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AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

WATER TRANSPORTATION VERSUS RAIL TRANSPORTATION
A SYMPOSIUM

THE VALUE OF WATER TRANSPORTATION

By Rufus W. Putnam,[1] M. Am. Soc. C. E.

Synopsis

The completion, within the past decade, of the slack-water improvement of the Ohio River, the connecting of the Great Lakes to the Mississippi River *via* the Illinois Waterway, and the near completion of the Lower Missouri and Upper Mississippi projects give rise to a series of questions for which approximate answers might be found. When this 3 500-mile system of trunk inland waterways is completed what will have been the cost to the public of these facilities for commerce and what will be a fair approximation of the annual fixed charges and of maintenance and operation costs? To what extent and in what manner has this business of water transportation developed? How much traffic will have to be developed in order to reimburse the public treasury for the expenditures made?

These and other questions of like application the writer endeavors to answer in an effort to satisfy his own mind as to whether there is any value to water transportation.

Historical

The regional plan of the Mississippi Valley, as it is known to-day, is the direct result of the necessity of using its rivers to transport persons and goods during the early period of exploration and development. Access to the great domains which the early explorer claimed in the name of his sovereign, was had by way of the national water routes then existing; while the development of that rich territory was almost entirely dependent upon the use of these routes by the traders. Thus, water transportation had an early and continuing influence upon the political and economic growth of a large portion of the area lying between the Allegheny and the Rocky Mountains.

Before the introduction of the steam engine as a means of propelling vessels, the rivers of the Middle West were navigated first by canoe, and, later, by keel boats, flat boats, and a few sailing vessels. The commerce of these early days was largely a down-stream movement because, with the keel boats and the flat boats, down-stream movements were made with the current and up-stream journeys by man power. Cargoes were shipped from Pittsburgh, Pa., for instance, to New Orleans, La., and there the boats were taken apart and sold for what they would bring as lumber. Sailing vessels never were a success for river navigation in those early days because the channels were not buoyed or otherwise marked, and groundings and more serious disasters were too frequent to warrant any extended use of this type of vessel.

A few years after the introduction of steam as a means of propulsion (about

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1810) the boats in use soon reached a degree of efficiency and reliability that permitted their being operated economically for up-stream as well as for down-stream traffic. The river settlements then began to increase in number and size; trade between various parts of the Mississippi Valley and the Gulf thrived; and New Orleans became the focal point of most of the domestic and foreign trade of the Middle West. By 1860, for example, there were about 3 600 arrivals and departures of river "packets" each year at that port, with more than 750 vessels of that type in commission on the Mississippi and Ohio Rivers.

While water transportation was thus in its prime and had already determined the plan of development of the Mississippi Valley, rail transportation was still in its infancy. This new method of transportation found the framework of the economic structure of the country fairly well outlined when it became strong enough to become effective; it is only natural, therefore, to find that the first railroads were initially feeders to river ports as well as to seaports. As a result, the river communities received a further impetus in their development which was reflected in increased commerce on the rivers and also in greater stability of the regional plan for the interior, based on the rivers themselves.

When the rail lines between the Middle West and the Atlantic seaboard were completed traffic which formerly was routed by river to New Orleans and thence by vessel to export trade and to Eastern ports was diverted from the river routes to the direct rail routes to the East, and what began as a battle between two groups of ports resulted in serious injury to water transportation in the interior. Shortly thereafter came the completion of rail lines located parallel to the river lines; the latter being faced with a competition that they could not meet, and their general demise was then only a matter of time.

During the greater part of this period of thriving river traffic, the physical improvements to the inland water routes of the Valley were limited to projects for canal construction, undertaken by some of the States, with some Federal assistance, and some canals built by private undertaking. The main arteries of commerce, the rivers, were left as Nature made them. Many sections of the rivers, therefore, were impassable during low water; channels were unmarked; and snags were a constant menace. Apparently, it was not until after rail transportation had gained the upper hand in the struggle for carrying the commerce of the Mississippi Valley that a sincere effort to avoid the inevitable was made on the part of interests which had considerable investment at stake. It is somewhat significant that the first comprehensive undertakings for the improvement of the major inland water routes were not begun until the decade of 1870 to 1880, when projects for the betterment of navigation conditions on the Mississippi, Missouri, and Ohio Rivers were adopted by the Congress of the United States.

Even had they been promptly and effectively carried to completion, these improvement programs were begun too late to save the "packet" system of river transportation. Outmoded and obsolete, it had no place in the modern system of moving goods and passengers, and was due for death regardless of what might have been done to prolong its life. Nevertheless, although this particular method of transportation became practically extinct, there must have been something of the nature of John Brown in it for "its soul went marching

on." Generations of men had made their living on the rivers, or were in some other ways directly dependent upon them; and they not only preserved the faith but they were determined that, somehow or other, river transportation would be returned to them if only the program of extensive improvement of navigation conditions could be completed. In this manner the river projects, launched well after the decline in the "packet" business had begun, were intermittently promoted until a new impetus came along and provided enough additional energy to bring them to their present state of near completion.

Navigation interests desirous of securing improvements on the Mississippi and Missouri Rivers were aided greatly in their efforts to obtain Federal appropriations for river works by two influences only partly related to water transportation. The great damages resulting from floods focused national attention on the general improvement of these streams from an entirely different angle, and, in addition, there was a desire to stabilize the banks which were quite subject to erosion during even intermediate and low-water stages. The alliance of the three interests involved in the control of these rivers has proved sufficient in recent years to insure the prompt execution and early completion of a program designed to serve them all.

The betterment to navigation conditions which has resulted from these programs of improvement to the rivers in the Mississippi Valley has permitted an efficient development of methods of water transportation which are entirely different from those of the packet-boat days. Passenger traffic (except for the excursion variety) has disappeared entirely. Freight traffic, which is now greater in volume and value than it was during the days when water transportation was the chief reliance of the commercial interests of the Mississippi Valley, is handled almost exclusively in large tows consisting of a number of barges adapted to carry the particular commodities which they are engaged in transporting, and pushed by towboats of ample power to handle up-stream as well as down-stream traffic. Naval architecture and marine engineering have made great progress with respect to the design of equipment suited to the various requirements of the water-transportation service. It is quite evident, therefore, that in treating the subject of present-day inland-water transportation in the Mississippi Valley one is dealing with facilities and equipment wholly different from those in use when the basis for the commercial and industrial framework of the interior was laid during the first half of the Nineteenth Century.

Existing Water-Transportation Facilities

Routes.—Of the 15 000 miles of rivers in the Mississippi Valley, said to be susceptible of improvement, only the main north and south and east and west arteries are under consideration in this paper. These arteries include the Mississippi River below Minneapolis, Minn., the Illinois Waterway from Chicago, Ill., to the Mississippi; the Ohio River; and the Missouri River below Kansas City. These comprise about 3 500 miles of completed or projected 9-ft channels. It is believed that all the more important factors involved in the problems affecting inland water transportation will be developed by thus limiting the scope of discussion. Improvements and operations on the tribu-

taries are not sufficiently different from those on the main routes to justify their inclusion. A notable exception is to be found in the case of the improvement of the Tennessee River System by the Tennessee Valley Authority, but the basis of that project, as it is now (1937) being undertaken, is so fundamentally different that it is in an entirely separate category from that into which the projects herein described naturally fall.

The total expenditures chargeable to capital improvements for navigation on these main routes amounts roughly to about $550 000 000 to date (1937); and it appears that approximately $100 000 000 will be required to pay for the works necessary to complete these projects. The cost of maintenance and operation has been estimated to amount to $10 000 000 per yr.

Terminals.—With the exception of the Great Lakes cities of Milwaukee, Wis., Detroit, Mich., Toledo, Ohio, Cleveland, Ohio, Buffalo, N. Y., and the inland City of Indianapolis, Ind., all the great cities of the Middle West are situated on this trunk system of inland waterways. As a result extensive facilities have been provided at many places of importance for the transfer and storage of freight handled over these navigation routes. Pittsburgh has extensive equipment for unloading coal from the Monongahela River; its steel mills have ample arrangements for loading steel on to barges destined for points on the Lower Ohio and Mississippi Rivers. Coal-loading points have been established at Colona, Pa., Huntington, W. Va., and Ashland, Ky., on the Ohio River, and terminals for the receipt of water-borne coal have been provided at Cincinnati, Ohio, Louisville, Ky., and Evansville, Ind. Facilities for handling steel, pipe, petroleum products, building materials, and miscellaneous cargo are available at some of the aforementioned points, as well as at Paducah, Ky., and at Cairo, Ill.

On the Lower Mississippi River, equipment is available at all the river cities of importance for the handling of water-borne commerce of all kinds including, in addition to those mentioned, cotton, sugar, sisal, bauxite, bulk cement, molasses, and grain. On the Illinois Waterway System, only recently completed, facilities are available for the loading of grain at many points, of miscellaneous cargo at Peoria, Ill., and at Chicago, Ill., and of coal at Havana, Ill. On the Upper Mississippi River municipal terminals have been provided at Davenport, Iowa, Rock Island, Ill., Dubuque, Iowa, and St. Paul and Minneapolis, Minn.; and Kansas City has recently provided itself with terminal equipment to serve the barges navigating the Lower Missouri River.

Substantial investments have been made at all these terminal points and at many others. The writer has no knowledge of any cumulative estimate of the cost of all these facilities; the mere recital of them should be sufficient, however, to give an indication of the comparative magnitude of some of the elements of the subject under consideration.

Transport Equipment.—A study of the latest published Government report on the subject shows that the commercial fleet in active operation during 1935, on the main inland water routes under consideration in this paper, consisted of about 540 towboats and 2 340 barges. The aggregate installed horsepower in the towboats approximated 176 000; the carrying capacity of the barges totaled about 1 600 000 tons. Details as to size and type of towboats

appear in Table 1. The replacement cost of this fleet (allowing $200 per 1 hp for towboats and $20 per ton for barges) approximates $67 000 000. More than 1 700 of the barges are built of steel.

TABLE 1.—CLASSIFICATION OF TOWING EQUIPMENT, MISSISSIPPI WATERWAYS

Size (indicated horse power)	Steam						Motor					
	Steel Hull			Wood Hull			Steel Hull			Wood Hull		
	No.	Indicated Horse Power		No.	Indicated Horse Power		No.	Indicated Horse Power		No.	Indicated Horse Power	
		Total	Average		Total	Average		Total	Average		Total	Average
More than 2 500	2	5 400	2 700	....	....	....	1	2 600	2 600	....	....	....
2 000–2 499	9	18 785	2 087	....	....	....	....	....	....	....	....	....
1 500–1 999	9	15 600	1 733	1	1 500	1 500	....	....	....	....	....	....
1 000–1 499	10	12 070	1 207	2	2 400	1 200	6	6 360	1 060	....	....	....
500–999	52	36 620	705	17	10 445	614	8	5 325	665	....	....	....
300–499	19	7 245	381	29	10 760	371	10	3 670	367	....	....	....
200–299	7	1 470	210	24	5 820	242	15	3 440	229	7	1 895	271
100–199	9	1 300	144	28	3 825	137	29	3 715	163	42	5 425	129
Less than 100	1	95	95	13	880	68	20	1 230	62	169	7 850	46
Total	118	98 775	837	114	35 630	311	89	26 340	296	218	15 170	70

In 1935, there were twelve operators, each having in active service one or more towboats with a total of at least 2 000 hp and barge fleets of 10 000 tons or greater carrying capacity. These operators had 87 towboats with an aggregate installed horse-power of about 85 000 and 937 barges with a total carrying capacity of 911 000 tons. About one-half the total equipment in active service was thus operated by twelve major transportation companies. The towboats in use by these companies had propulsion equipment averaging almost 1 000 hp and the barges in use averaged nearly 1 000 tons in carrying capacity. It is appropriate, therefore, to assume the use of units of these sizes, or greater, in a study of modern inland waterway transportation costs.

As with other methods of transportation the cargo carriers are adapted to the kinds of commodities carried. Barges for carrying coal have been largely standardized, and are built to dimensions best suited for easy passage through the locks on the waterways they are required to traverse. These are open top barges and are suitable for the transportation of any other type of dry bulk cargo which does not have to be protected from the elements. Sand, gravel, and crushed stone are usually carried on deck barges, to permit draining if wet, and also because the greater weight per unit of volume of this class of material results in the requirement of less cargo space to obtain project draft. Special tankers have been developed for the transportation of liquids of different kinds, although in some cases liquid cargo is carried in side and bottom tanks built into barges primarily designed for carrying miscellaneous cargo.

One of the barge lines which transports more miscellaneous cargo on inland rivers than any other operator has been prominent in the development of the modern type of barge for the transportation of freight of that character. These barges have been standardized in two sizes. For operations on the Lower

Mississippi River, where a full 9-ft channel is available, barges designed to carry 2 000 tons are used extensively. (Each barge carries 2 000 tons of freight with a draft of 8 ft. The average running time for a tow, including all stops, is 8 days from St. Louis to New Orleans, and 15 days on the return trip.) The hull of the barge may be used for grain or for miscellaneous cargo; the cargo house which is built on top of the hull is also used for package freight of various kinds. Hatches and side doors have been standardized as to dimensions and as to spacing, and correspond closely with the door spacing at the terminals. Barges in this trade on the Upper Mississippi River and into Chicago (where vertical clearances are controlling) are similar in design, but built on a smaller scale.

Self-propelled barges are somewhat rare on the Mississippi System of waterways; quite contrary to practice on the canals and other waterways in the East. One barge line had three in service as express boats for several years, operating between St. Louis and New Orleans. They were self-contained units carrying about 1 600 tons of high-class freight on a draft of 7 ft. Although they made fast time in navigating, too much time was lost at the terminals, with the result that they were taken out of service in 1935.

Traffic.—Without going into details it should be sufficient for the purposes of this paper to state that the traffic carried on each component of this main system of inland waterways has been much greater in recent years than during the period when water transportation was the only means of transporting large quantities of freight over long distances. Furthermore, there has been substantial growth of traffic since the projects for the improvement of these routes have been nearing completion. Excluding the traffic on the tributaries, the total water-borne commerce on this system in 1915 approximated 15 000 000 tons; it now (1937) totals about 30 000 000 tons per yr. A large proportion of this movement is over a comparatively short haul and is composed of coal and building material on the Ohio River. The average haul has been gradually increasing, but whether this will continue is problematical, as an important factor in this tendency has been the rapid development of long-distance traffic, during the past 10 to 15 yr, over the Mississippi River routes.

Water-Transportation Costs

There has been sufficient experience with reference to the cost of constructing and operating water-transportation equipment since about 1920 to make it possible to reduce this experience to useful data for those interested in the subject.

Equipment Costs.—Although there appears to be a wide variety in costs of towboats this is due largely to differences in size or class of operating machinery. The more modern type of steam towboats will be found to vary in cost, roughly, as follows:

Range of indicated horse power	Cost, in dollars, per indicated horse power
500 to 1 000	250
1 000 to 1 500	225
1 500 to 2 000	200

Towboats using Diesel engines are usually slightly higher in cost. The foregoing data apply to installations with direct drive; if electrical drive is provided, the costs are higher. There is also a wide range in the costs of barges, but they may be grouped, roughly, as follows:

Type of barge	Cost, in dollars, per ton of cargo capacity
Hopper type	15 to 20
Deck barges	18 to 25
Package freighters	30 to 40

Fixed Charges.—An approximate idea of the fixed charges which are usually applied to barge operations may be gained by reference to Table 2.

TABLE 2.—FIXED CHARGES IN BARGE OPERATION, AS PERCENTAGES OF THE TOTAL COST

Description	Towboats	Barges	
		Bulkers	Package freighters
Interest	6	6	6
Depreciation	5	6	4
Insurance	3	2	2
Taxes	2	2	2
Annual overhaul	2	2	2
Total fixed charge*	18	18	16

* Percentages of total cost.

Operating Costs.—Such costs include the items of wages, food, fuel, lubrication, repairs to towboats, repairs to barges, superintendence, and miscellaneous.

The wage item depends upon the size of the crew employed and also the locality in which the operations are conducted. One barge line pays an average of about $3 per man per day. Its operations extend throughout the entire stretch of the Mississippi River below St. Paul, and the crews on the towboats vary in number between 20 and 30 men. In the Pittsburgh District wages have averaged as high as $6 per man per day. The operators in that vicinity, however, have been able to use fewer men in the crews (normally, about twenty men on a towboat with a capacity of 1 500 ihp). Rates on the Upper Illinois River vary between $4 and $5 per man per day, and the crews more nearly correspond in number with those in the Pittsburgh District.

Subsistence costs vary between $0.55 per man per day for the barge line mentioned to as high as $1.10 per man per day in the Chicago District. Pittsburgh rates are in the vicinity of $0.75 per man per day. These rates depend upon the number of men employed and upon the availability of good markets.

Fuel costs vary in different parts of the country. In the Pittsburgh District coal is generally cheaper than fuel oil, whereas on the Mississippi and Illinois Rivers fuel oil has the advantage. Fuel oil requirements are best expressed in ton-miles of cargo transported per barrel of fuel. For steam towboats this factor averages about 2 600 ton-miles per bbl of fuel for oil-burning steamers and about 4 800 ton-miles per bbl of fuel for Diesel engines.

These performance data apply to operations in still water where ample depths are available.

Lubricating costs for steam towboats average approximately 3 mills per ihp for each full day of service. For towboats equipped with Diesel engines the cost of lubrication is about 8 mills per ihp per day for a two-cycle engine and about 4 mills per ihp per day for a four-cycle engine. These values are based on the use of a centrifuge for cleaning the oil and a price of $0.60 per gal.

Part of the total cost of repairs to towboats is included as a fixed charge under the caption, "Annual Overhaul." In addition, repairs throughout the season for steam towboats cost on an average of $0.03 per ihp per day and, in the case of Diesel engine installation, this cost is about $0.04 per ihp per day.

The cost of repairs to barges depends largely upon the service for which they are used. If used for handling bulk cargo, which is unloaded by heavy machinery, the total annual costs are comparatively higher than in the case of package-freight barges on which lighter materials are unloaded by hand. These two services represent the extremes, and repair costs will be found to vary, therefore, between 1% and 5% of the first cost of the barges.

Costs of superintendence vary with the character of service and the territory covered. A check of the various operations indicates that this item will amount annually to from 1% to 2% of the cost of the fleet. Miscellaneous charges include deck supplies, laundry, waste, and other small items and are found equal to $0.01 per ihp per day of service.

Performance.—A study of performance statistics of a large number of steam towboat operations on the Mississippi River gives the ton-miles per horse-power-hour, up stream, as 10; and down stream, 17. These values will be found to vary with the project depth and with the quantity of free water under the barges. If the barges are "hugging" the bottom, there is considerable drag, which is reflected in decreased speed for the same fuel consumption. For instance, in still water, performance factors would vary with the depth of water under the barges, as follows:

Depth of water under the barges, in feet	Performance, in ton-miles per horse-power-hour
5	15
2	12

Transportation Costs Consolidated.—Table 3 contains the estimate of costs for the several items involved in connection with the operation of a typical tow of 5 000 tons propelled at the average rate of $5\frac{1}{2}$ miles per hr in still water. Costs are estimated for different load factors applying to the propelling units and also for several load factors applying to conditions under which barges might be utilized. Table 4 gives the cost of transportation for self-propelled barges of three different sizes and under varying conditions as to load factor. The costs for the 1 600-ton unit were compiled from records of actual operations; those for the 2 400 and 3 200-ton units were computed for purposes of comparison.

Representative values have been used for unit costs of fuel and lubricating oil, and for wages and subsistence, and a suitable allowance has been made wherever necessary to correct for partial use of equipment. Referring to

TABLE 3.—COSTS OF WATER TRANSPORTATION BY STEAM TOWS

(Capacity, 5 000 tons; 1 500 indicated horse power; and speed of 5.5 miles per hour in still water)

Description	Load factor of barges (percentage)	ANNUAL COST, IN THOUSANDS OF DOLLARS*								Millions of ton-miles per year	Unit cost, in mills per ton-mile
		First cost of unit	Fixed charge	Bargemen's wages	Wages* and food on towboat	Maintenance and repair of barges	Fuel, lubrication, and maintenance and repair of towboats	Superintendence and miscellaneous	Total		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)
(*a*) TOWBOAT LOAD FACTOR, 80 PER CENT.											
Bulk cargo......	20	700	126.0	9.0	32	8.0†	70.5	9.0	254.5	158.4	1.613
	40	500	90.0	4.5	32	6.6†	70.5	8.0	211.6	158.4	1.336
	60	440	79.2	3.6	32	5.3†	70.5	7.0	197.6	158.4	1.248
Package freight ..	20	900	150.0	9.0	32	6.0‡	70.5	12.0	279.5	158.4	1.765
	40	600	102.0	4.5	32	5.5‡	70.5	11.0	225.5	158.4	1.425
	60	510	87.6	3.6	32	5.0‡	70.5	10.0	208.7	158.4	1.318
(*b*) TOWBOAT LOAD FACTOR, 70 PER CENT.											
Bulk cargo......	20	620	111.6	7.2	32	6.4	63.5§	8.1§	228.8	138.6	1.649
	40	460	82.8	3.6	32	5.6	63.5§	7.2§	194.7	138.6	1.404
	60	420	75.6	2.7	32	4.8	63.5§	6.3§	184.9	138.6	1.333
Package freight ..	20	780	130.8	7.2	32	4.8	63.5§	10.8§	249.1	138.6	1.797
	40	540	92.4	3.6	32	4.5	63.5§	9.9§	205.9	138.6	1.484
	60	480	82.8	2.7	32	4.2	63.5§	9.0§	194.2	138.6	1.400
(*c*) TOWBOAT LOAD FACTOR, 50 PER CENT.											
Bulk cargo......	20	580	104.4	6.3	32	5.6	49.4‖	6.3‖	204.0	99.0	2.060
	40	440	79.2	3.6	32	4.0	49.4‖	5.6‖	173.8	99.0	1.756
Package freight ..	20	720	121.2	6.3	32	4.2	49.4‖	8.4‖	221.5	99.0	2.238
	40	510	87.6	3.6	32	3.5	49.4‖	7.7‖	183.8	99.0	1.856
(*d*) TOWBOAT LOAD FACTOR, 30 PER CENT.											
Bulk cargo......	20	460	82.8	3.6	32	3.2	35.4¶	4.5¶	161.5	59.4	2.718
Package freight ..	20	540	92.4	3.6	32	2.4	35.4¶	6.0¶	171.8	59.4	2.892

* 300 days in commission. † 2% to 4% of Column (3). ‡ 1% to 3% of Column (3). § 90% of the corresponding values in Table 3(*a*). ‖ 70% of the corresponding values in Table 3(*a*). ¶ 50% of the corresponding values in Table 3(*a*).

Table 3(*a*), Column (3), the first cost includes: Towboats @ \$300 000; bulk cargo barges @ \$20 000; and package-freight barges @ \$30 000. In Column (4), the annual fixed charge includes: Towboats and bulk cargo barges @ 18%, and package-freight barges @ 16 per cent. Towboat wages (Column (6), Table 3(*a*)) are based on an average rate of \$1 350 per yr per man; and, bargemen's wages (one man for each two barges) on a rate of \$900 per yr. The

assumed food allowance for the towboat crew (Column (6)) is \$250 per yr per man. Fuel and lubrication costs (Column (8), Table 3(*a*)) amount to \$36 per yr per ihp, with the cost of maintenance and repair at \$11 per yr per ihp. The costs in Column (9), Table 3(*a*), are based on an assumption of \$20 to \$30 per day for bulk carriers and \$30 to \$40 per day for package freighters. Table 3(*a*) is of no final use in itself but may be used in the process of computing water transportation costs if the necessary load factors are known. Unit costs are shown in Fig. 1.

TABLE 4.—COSTS OF WATER TRANSPORTATION, SELF-PROPELLED BARGES
(Speed in still water, 8 miles per hour)

Size, in tons	Indicated horse power	ANNUAL* COST, IN THOUSANDS OF DOLLARS										Millions of ton-miles per year	Unit cost, in mills per ton-mile
		First cost of unit	Annual fixed charges @16%	Wages†	Commissary‡	Lubrication§	Fuel‖	Maintenance and repair¶	Superintendence	Miscellaneous**	Total		
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)	(14)
(*a*) LOAD FACTOR, 80 PER CENT.													
1 600	800	208	33.3	24.3	4.5	0.8	28.0	7.2	6.0	2.4	106.5	73.8	1.443
2 400	950	262	41.9	24.3	4.5	1.0	33.2	8.6	6.0	2.9	122.2	110.6	1.106
3 200	1 100	316	50.6	24.3	4.5	1.1	38.5	9.9	7.5	3.3	139.7	147.5	0.947
(*b*) LOAD FACTOR, 70 PER CENT.													
1 600	800	208	33.3	24.3	4.5	0.7	25.2	6.5	8.4		102.9	64.5	1.596
2 400	950	262	41.9	24.3	4.5	0.9	29.8	7.7	8.9		118.0	96.8	1.219
3 200	1 100	316	50.6	24.3	4.5	1.0	34.7	8.9	10.8		134.7	129.0	1.043
(*c*) LOAD FACTOR, 60 PER CENT.													
1 600	800	208		62.1			28.8		8.4		99.3	55.3	1.794
2 400	950	262		70.7			34.1		8.9		113.7	82.9	1.372
3 200	1 100	316		79.4			39.6		10.8		129.8	110.6	1.173
(*d*) LOAD FACTOR, 50 PER CENT.													
1 600	800	208		62.1			25.2		8.4		95.7	46.1	2.077
2 400	950	262		70.7			29.9		8.9		109.4	69.1	1.583
3 200	1 100	316		79.4			34.7		10.8		124.8	92.2	1.356
(*e*) LOAD FACTOR, 40 PER CENT.													
1 600	800	208		62.1			21.6		8.4		92.1	36.9	2.500
2 400	950	262		70.7			25.6		8.9		105.2	55.3	1.903
3 200	1 100	316		79.4			29.7		10.8		119.9	73.7	1.625

* 300 days in commission. † At \$1 350 per yr per man; 18 members in the crew. ‡ At \$250 per yr per man. § At \$1.00 per yr per ihp. ‖ At \$35 per ihp. ¶ At \$9 per ihp. ** At \$3 per ihp.

Load Factors.—A load factor in water transportation might be said to be the ratio between the total number of ton-miles resulting from one season's operation of a unit and the theoretically possible total movement. Therefore, to

determine a load factor for a towboat it is first necessary to determine the total number of hours during which the towboat will be in commission. Multiply this by the number of tons in a capacity tow and by the still-water speed, in miles per hour; the result gives the theoretical total tonnage possible for the number of hours during the year in which the towboat is in commission. The actual performance will be measured by the number of hours the towboat is in line service, deductions having been made for terminal and mechanical delays and also for channel delays, such as groundings, time lost through passing through bridges or locks, etc. Another item that must be used to reduce the load factor for a towboat is the average loading of carriers as compared with the theoretical capacity load. The final item for consideration is the character of the cargo movement, itself; if it is a balanced movement no further adjustment in load factor is required; if the movement predominates in one direction correction must be made accordingly.

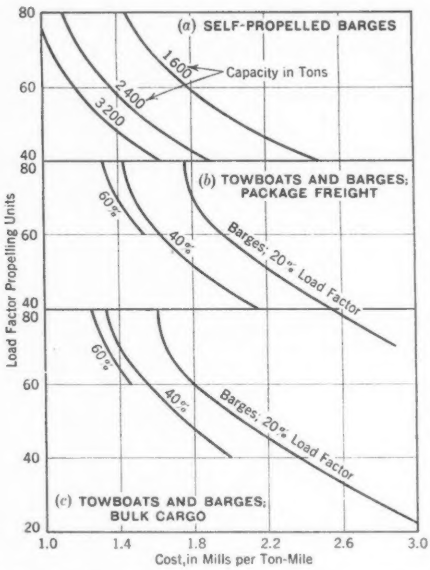

FIG. 1.—UNIT COSTS OF WATER TRANSPORTATION

In the case of barges a similar procedure should be followed in determining the load factor for the operation under consideration. It will be seen that a very thorough knowledge of operating conditions must be available before these load factors may be determined intelligently.

IN JUSTIFICATION

Preliminary.—Many notable papers have been published in an attempt to prove economic justification for many of the waterway projects that have been undertaken in the United States. Possibly as many contributions have been made on behalf of rail transportation. The waterway papers have been discussed and criticized by railroad advocates, while the railway point of view has been argued against by those favoring the waterways. One would think that all points for both sides of the controversy had been covered time and time again; that there was nothing more to be said, and that nothing could be gained by prolonging the argument; and yet the strife appears to continue; waterways are accused of "robbing" the taxpayer and of taking traffic away from their competitors, whereas the railroads are alleged to be engaged in the nefarious practice of reducing rates wherever water competition exists and making up the loss at the expense of their customers located where the advantages of water competition are not available. Of course, this attitude is understandable when it is realized that each side is attempting to protect its own interests; but the results are not necessarily along the lines of better understanding or of constructive accomplishments for the public at large.

Tangible Economic Values.—The main water transportation routes in the Mississippi Valley will have cost the United States at the rate of about \$186 000 per mile of length when completed. Maintenance, including the operation of the locks and movable dams, will cost at the annual rate of approximately \$3 000 per mile. The capacity of these water highways for carrying freight will vary. The open rivers will be able to carry practically an unlimited tonnage, but the canalized sections will be limited by the rapidity with which the carriers may pass through the locks of highest lift in those sections. For instance, the traffic from the Lower Mississippi River and the Missouri River to and from the Upper Mississippi and the Illinois Waterway System will be limited by the capacity of the lock on the Mississippi River at Alton, Ill.; through traffic between Chicago and the Valley may not exceed the capacity of the lock at Lockport, Ill., at the lower end of the Chicago Drainage Canal; and the capacity of the Upper Ohio River will be determined by that of the locks at the dams below Pittsburgh. With single locks at these three critical locations, as built or as proposed, the limits will approximate between 20 000 000 and 30 000 000 tons per yr, respectively. The construction of twin locks at these places would result in more than doubling the capacity of those sections of the waterways at a cost which would be comparatively small.

By way of comparison, it is understood that modern railroad construction costs in the Mississippi Valley vary between \$50 000 and \$70 000 per mile of single track, and that maintenance of way and structures cost on the average of from \$2 000 to \$3 000 per mile annually under heavy traffic. Furthermore, it is the understanding that the annual traffic capacity of a single-line railroad varies between 20 000 000 and 30 000 000 tons, depending on the class of freight which predominates. It is thus apparent that inland-waterway construction is much less flexible than is the case with railroad construction and, without regard to capacity, it costs more to build and to maintain. Furthermore, railroad distances between given points are generally shorter than those by water, which is an additional credit to railroad transportation when construction and maintenance costs are under consideration.

Conditions vary so much as to terminal operations that it is difficult to draw any general conclusions. Usually, rail terminals are located on more expensive property than water terminals so that total fixed charges are comparatively high; but since they are likely to have a higher use factor there is probably little difference between unit fixed charges. With operating costs conditions should favor the railroads because such transfer operations are generally conducted at one elevation, whereas at the river terminals there is a wide fluctuation between low and high-water stages which requires a considerable vertical movement of cargoes. Where joint rail and water transport operations are involved additional transfer operations are required, and as regards the waterway traffic in the Middle West, these transfers are always at the cost of the waterway carrier. This condition is somewhat different from the methods in vogue at several seaboard rail terminals where such costs are absorbed by the rail carriers.

As to comparative costs of the two methods of transportation, there should be no further cause for misunderstanding. Modern methods of inland water transport have been successfully in operation for a sufficient length of time, and

enough data have been assembled on the subject so that fairly definite conclusions may be reached. First costs and fixed and maintenance charges for equipment for both types of transportation facilities on comparable operations are readily ascertainable, and actual operating costs for a variety of traffic conditions are known. Why argument continues along these lines is a question.

A study of various authorities on the subject indicates that for railroad operation in the Middle West the cost of maintaining equipment plus fuel, supplies, and wages, averages very close to 4 mills per ton-mile. Interest on investment in equipment and its depreciation (which are dependent upon the load factor when unit costs are under consideration) vary from about $0.0006 per ton-mile for a load factor of 100% to as much as $0.0015 per ton-mile when the load factor is 40 per cent. Total unit costs for this range of load factors, therefore, lie between $0.0045 and $0.0055 per ton-mile. Comparable costs for water transportation, as demonstrated previously, are from $0.00125 to $0.003 per ton-mile. The latter should be corrected for circuity as the water distances are generally longer than those by rail. Between points on the Ohio River this factor averages about 1.4; between Chicago and St. Louis, it is about 1.2; on the Lower Mississippi River, it approximates 1.5; and up stream from St. Louis the factor is 1.1.

Comparison of Costs.—It is now possible to make a rough comparison to determine the net tangible cost to the country at large of supporting traffic on the main inland waterways of the Mississippi Valley. If it is assumed that all the existing waterway traffic was carried by railroads serving the same points and allowing a cost of $50 000 per mile for the roadway, a capital investment of about $175 000 000 would be involved compared with the present cost of the water highways of $550 000 000. The annual traffic to be handled is about 5 500 000 000 ton-miles *via* water routes, the equivalent of about 4 000 000 000 ton-miles by rail. Comparative costs would then be as shown in Table 5(*a*). It

TABLE 5.—COMPARISON OF COSTS, IN MILLIONS OF DOLLARS

Description	(*a*) Water Traffic Carried by Railroads Serving the Same Points		(*b*) Comparison on the Basis of Present Waterway Improvements		(*c*) Comparison if Waterway Traffic Equals the Capacity of a Single-Track Railroad	
(1)	Rail (2)	Water (3)	Rail (4)	Water (5)	Rail (6)	Water (7)
Interest	17.5	44.0	35	52	17.5	52.0
Maintenance of way and structures	4.0	8.0	36	15	25.0	12.5
Operating charges	22.0	13.75	162	100	112.5	78.8
Total	43.5	65.75	233	167	155.0	143.3

is apparent, therefore, that the country at large is paying more for its transportation in the amount of about $22 000 000 per yr on account of the present use of the main inland waterway routes in the Mississippi Valley.

When improvements under way are completed these water transportation routes could easily accommodate an annual traffic of at least 50 000 000 000 ton-

miles and with the addition of duplicate locks at critical locations, this capacity could be doubled. If a comparison is made on the basis of the capacity of the present waterway improvement projects (in which case the first cost of railroad roadway to carry it would be twice the sum given previously) costs involved would approximate those in Table 5(*b*). In this case the comparison favors water transportation. If the water traffic totaled 35 000 000 000 ton-miles the corresponding railroad tonnage would be about 25 000 000 000, approximately the capacity of a single-line road. Under such conditions the comparison in Table 5(*c*) would apply. The future tangible economic justification of this large expenditure for waterway development, therefore, lies in the ability of industry and commerce in the Middle West to develop and support water traffic approximating 35 000 000 ton-miles per yr.

It is to be noted that these comparisons take no account of rates charged for either rail or water transportation. Rates are notoriously artificial as far as the railroads are concerned and where they are in competition with water they bear little relation to the cost of services performed. Water rates, on the other hand, are generally as close below the corresponding rail rates as the traffic will bear. In any event it is the costs that determine the net direct effect on the national wealth, and it is only when rates become so low as to be destructive of healthy competition or so high as to prevent industrial development that they affect a drain on national resources.

The foregoing comparisons of costs of transportation were intended to be made in a way that would avoid controversial questions. The results show the tax on the nation's pocket-book of the existing uncompleted system of waterways as reflected in transportation costs. Questions of subsidy to water carriers are thus eliminated; nor is consideration given to the subsidy enjoyed by rail lines paralleling the water routes paid for by shippers located where they cannot use them.

In one respect only are the comparisons incorrect; that is, with regard to the costs of the transportation routes themselves. For the water routes, the total costs result from a study of the records of actual expenditures by the Federal Government since the inception of the projects. These costs, therefore, are not replacement costs, but the accumulation of expenditures from sporadic appropriations made over a long period of years, a substantial proportion of which has been undoubtedly wasted by reason of the uncompleted nature of the construction works. For the rail routes replacement costs are used and, therefore, they make no allowance for wasteful expenditures of the past which are the experience of many great enterprises. There is no accurate method of arriving at a replacement cost of the waterway system; nor is there any way of telling how much has been spent to bring any given part of the railroad system of the country to its present efficiency and condition. In this respect, when the conclusion is drawn that inland water transportation over these particular routes costs the nation approximately \$22 000 000 per yr, the rail carriers are given a substantial advantage.

Intangible Values.—In a discussion of the intangible effects of a major program of public improvements as involved in the development of water-transportation routes in the Mississippi Valley, it is necessary at the beginning

to take as broad a point of view as possible. The investigator is concerned neither with the immediate effect on small communities nor with individual enterprises. Water carriers as well as rail carriers are only a means to an end, and that end is best expressed by the statement that the people still are engaged in building a nation.

Another point to be kept in mind is that most of the construction operations in connection with the improvement of these routes have been completed and the expenditures already made. The sum of $550 000 000 has been expended, 2 500 miles of channel finished, and about $100 000 000 will be required to complete the remaining 1 000 miles of channel to project dimensions. Arguments as to the wisdom of works already completed are somewhat academic as the money expended cannot be recovered without putting these projects to work. It would seem more to the point for all concerned to join efforts to insure as large a return on the previous investment as possible, whether it was good or bad.

There is little question but that the old "packet" boat was a controlling influence in the formation and growth of the "plan" for the Middle West. As a result, this "plan" was based on the great rivers, and as many as have been the changes in industrial development and in the forms of transportation which serve both commerce and industry, the fundamental elements of that "plan" still persist. The present problem, therefore, is simply to determine whether that "plan" has sufficient merit to justify its perpetuation as against any other that might be advanced.

All the great nations of the world have been developed on the premise that the sea is the base of the system of transportation. The world ports achieved their pre-eminence because they served as focal points for the lines of transportation to the interior and as gateways for commerce over the free routes of the high seas to domestic as well as foreign destinations. The controlling elements in this situation are: (1) That ocean transport was, and still is, cheaper than land carriage; and (2)—which is possibly of greater importance—that the water highways are free and dedicated to the unrestricted use of all.

By close analogy the interior of a great country may be developed to best advantage by preserving for the use of the general public the free water highways of its great rivers even at some expense, if necessary. The principle involved is identical. In the writer's opinion the price of about $22 000 000 per yr is a small one for this country to pay if, in making this investment, it accomplishes the multiple purpose of salvaging large expenditures made in previous years, of preventing a violent readjustment in the distribution of industries and population, and of preserving a group of minor "sea bases" located on a secondary coast line 7 000 miles in length.

There is another important set of intangible values involved in this question which can be emphasized to best advantage by quoting a short part of a statement[2] by the late C. E. Grunsky, Past-President, Am. Soc. C. E.:

"In his opening paragraph the author [the late William Murray Black, M. Am. Soc. C. E.] presents the conclusion that a new waterway line of transportation should not be established unless the need for it can be shown and

[2] *Transactions*, Am. Soc. C. E., Vol. 88 (1925), p. 553.

unless 'if established, it will produce an annual saving in the cost of transportation greater than the interest on construction plus maintenance and operating costs.' There is another element which should sometimes be considered when the advisability of such a line of transportation is being studied, and by virtue of which many a project may be found advisable, despite the fact that it cannot, perhaps, for many years in the future, produce revenue to meet in full the interest charges against it. This element is the project's contribution to general prosperity as measured in terms of population, of business, and of property values. It becomes a factor because the expenditure of funds in its construction, and the utilization of the facilities which it provides when it is completed, bring increase of population; and to the extent that the growth in population of the zone coming under the influence of the improvement is thus accelerated, to the same extent will land values increase and wealth be created, which may fairly be weighed against the cost of the improvement when the advisability thereof is under consideration. Society, in other words, may reap material advantage from many an enterprise that would be condemned by the stockholder who weighs only the cost of the service against the prospective revenue."

The application of the principle stressed in this quotation to the advisability of completing and utilizing the main water-transportation routes of the Mississippi Valley should not encounter serious opposition.

THE HIGH COST OF INLAND WATER TRANSPORTATION

By S. L. Wonson,[3] M. Am. Soc. C. E.

Synopsis

Due to its historical background, water transportation in the Mississippi Basin has, in earlier years, impressed itself strongly into the national consciousness. After the decline of river traffic, following the fixation of traffic routes and the development of railways, having certain elements of inherent superiority in satisfying traffic needs, this early impression resulted in a public demand for the continuing development of river transportation at public expense.

This demand was largely based upon the supposed necessity of waterways to supplement inadequate facilities of the railways and to act as regulators of their rates and practices. Under present and foreseeable future conditions, these reasons have lost such validity as they may have had, and the justification of waterway development in the Mississippi Valley at the expense of the taxpayers now rests upon the test of relative economy of water and rail transportation, taking into account all elements of cost.

The rivers of the Valley are not of the class of waterways freely navigable in a state of Nature; they require large expenditures for channel improvement. The major elements in the cost to the public of transportation on these rivers are: The annual charges pertaining to Federal expenditures for improvement, and the service charges of the river carriers. Certain other elements of cost, such as those pertaining to bridges and river terminals, are not now known in total.

When the two major elements are reduced to a ton-mile basis, for comparison with the total cost to the public of rail transportation, which is closely known, it is found that they generally exceed the latter, in many cases to a marked degree, and that any impression as to superior economy of river transportation in the Valley is based upon a misapprehension.

Introduction

The question of waterway improvement for transportation purposes, particularly in the great interior basins of the United States, has been long and actively discussed, before this Society and elsewhere, and it would appear that every pertinent viewpoint has been adequately covered. The continued activity of the question, however, would seem to indicate that a realistic appraisal, from the viewpoint of the public interest, has not yet acquired general accept-

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ance, and that river improvement for transportation purposes has been and is being over-emphasized, to the detriment of the primary function of these streams, namely, that of drainage channels.

Viewpoints

The rivers of the Mississippi Basin exercised an essential function in the national life from the time of the first explorers in the Sixteenth Century, through the years of the pioneers and settlers, and well into the period of settled trade and commerce that opened with the Nineteenth Century. As long as the muscles of men and animals were the only motive power available, river transportation held an inherent superiority in efficiency over any form of land transportation.

Consequently, there were impressed into the national consciousness certain feelings as to the superiority and virtue of river transportation as such, and as to the necessity of its continuance, in the future as in the past, as an essential part of the national life. It is a characteristic of such general impressions, upon other subjects as well as this, to persist for some time after their factual basis has passed, and to be evidenced, in discussion of the particular subject, by emphasis upon the emotional, rather than the realistic, viewpoint. The Mississippi Valley Committee of the Federal Public Works Administration, for example, writes of the waterways in the Valley, "they have a romantic and traditional interest which cannot be disregarded."[4]

Decline of the Rivers

The function of the railroads as freight carriers did not develop immediately upon their first construction. Like the first river steamboats, the first railroads were primarily passenger carriers. An examination of the railroad map of 1840 will disclose the complementary relation of the early railroads to the waterways. West of the Alleghenies the first railroads were mostly short lines running to the Great Lakes and to various rivers. The same situation is marked on the railroad map of 1850. The map of 1860 shows a different picture. The shift of traffic from river to rail had begun, following the great change in direction of traffic initiated by the opening of the Erie Canal. The growth of the country was such, however, that neither of these causes produced an immediate decline in Mississippi River commerce; the year 1860, for example, exceeded all previous records in number of steamboat arrivals and value of goods received by river at New Orleans, La.

Professor Frank H. Dixon cites[5] two causes, independent of rail competition, for the rapid decline in Mississippi traffic after 1860: (*a*) The Civil War, which closed the lower river to commerce for several years; and (*b*) conditions at the mouth of the river which, after the introduction of steam navigation, seriously obstructed the access of ocean carriers to the Port of New Orleans, and which were not remedied until the completion of the Eads jetties in 1877.

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[4] Rept., Mississippi Val. Committee, PWA, October 1, 1934, p. 4.

[5] "A Traffic History of the Mississippi River System," by Frank Haigh Dixon, *Document No. 11*, U. S. National Waterways Commission, 1909, Govt. Printing Office, Washington, D. C.

Natural Disadvantages

In comparison with the service which the rail carriers were in a position to offer, some of the natural disadvantages of the rivers of the Valley as transportation routes may be mentioned as:

(1) Rigidity of location, preventing a universal and flexible service, requiring use of land transportation to serve interior points and involving additional time and expense for transshipment;

(2) In many cases the river courses lie across, rather than along, the major currents of traffic movement;

(3) Circuity, or greater distance between given points;

(4) Channel conditions, imposing limitations upon speed of movement and upon use of economical types of craft, and only to be partly remedied by large expenditures;

(5) Large variations in river level, requiring additional elements of dock and wharf facilities for handling goods between river bank and river craft; and,

(6) Closure of northern waterways during the winter season.

The rail carriers, in turn, held an inherent superiority over the rivers in transportation efficiency. This situation seems not to have been as clear prior to 1870 as it is in retrospect. Professor Dixon makes the interesting statement that:

"* * * one of the causes assigned for the building of cars by shippers was the fear of the railways that the restoration of river business after the war would have such a serious effect upon their business that it would be unwise for them to make the necessary outlay themselves."

Rivers Needed to Supplement Railroads

The shift of traffic from river to rail, and the rapid growth of the country and of its transportation needs after the Civil War, caused the railroad plant to be overtaxed at times and the situation of the waterways from the viewpoint of the public interest became the concern of various official committees and commissions, as well as of many private organizations and individuals. All these bodies based the need for inland waterway transportation upon the periodic inadequacy of the railroad plant as then functioning, and the national necessity for supplementary and additional transportation facilities.

The sudden and large increase in the nation's commerce and transportation needs during the World War again taxed the railroad plant to its limit, and, together with the taking over, by the United States, of transportation on certain waterways in 1918, apparently contributed to the feeling that land transportation could not be considered adequate for the nation's needs; that it must be supplemented by inland waterway development.

As the march of events and the passing of an era came to discount the correctness of this conclusion, emphasis seems to have shifted to a superior economy claimed for inland waterway transportation.

Economy of River Transportation

From the viewpoint of economy, waterways fall at once into two distinct classes: First, the oceans, the wide and deep estuaries of great rivers, and certain

great lakes (including, in some cases, relatively short connecting channels); and second, rivers, such as the rivers of the Mississippi Basin, which have the natural disadvantages previously mentioned and which require large expenditures to permit navigation by a restricted type of craft. Waterways of the first class present no great limitation upon efficient type, size, and operation of the cargo carrier, and constitute transportation routes superior in economy to any present form of land transportation.

From the viewpoint of transportation economy no analogy can follow from one class of waterway to the other, unless there be considered and included the cost of creating and preserving such conditions of the rivers as navigation upon them may require.

Of the total costs of river improvement only a part is recorded so as to be available, namely, that part representing the direct expenditures of the United States and, in some cases, of political subdivisions. Substantial additional costs, such as those involved in the construction and reconstruction of highway and railroad bridges, combine to make up the true but unknown total. A finding as to the various elements of true total cost pertaining to public improvement work, including river improvement, was adopted by the American Railway Engineering Association in 1935[6] and is also stated in the 1935 report of the Committee of the Engineering-Economics and Finance Division on Principles to Control Governmental Expenditures for Public Works.[7]

Rivers as Regulators of the Railroads

Accompanying the thought that waterway development is necessary to provide additional transportation facilities, has been the viewpoint that improved waterways are natural competitors of the railroads and effective regulators of their rates. This theory was advanced as early as 1874 by the Windom Select Committee on Transportation Routes to the Seaboard of the United States Senate. The more clear-sighted commentators, however, have recognized the fallacy of this view.

In 1909, Professor Dixon stated that "if the purpose is to reduce railway rates, there are more direct and less costly methods of accomplishing this result." In its first report of 1910, the National Waterways Commission pointed out that expenditures on waterway improvement for the purpose of regulating railroads or reducing rates were not justified by either reason or experience.

The scope of railroad regulation has been so extended and strengthened in recent years as to justify the conclusion of these two authorities. The subject is treated by the National Transportation Committee, which, in its report of 1933 to the President, states, in part:

"The development of regulation and of new methods of transport make it unnecessary for Government further to create and foster competition with or among railroads as a defense against monopoly. That is an expensive and ineffective attempt to do indirectly what Government has shown its ability to do directly. Regulation is sufficient."

Regulation of the railroads "has been practiced long enough and sufficiently extended to prove that it dominates competition or any other influence as the

[6] *Proceedings*, A. R. E. A., Vol. 36 (1935) p. 238; also, *Bulletin 371*, November, 1934, p. 238.

[7] *Proceedings*, Am. Soc. C. E., February, 1936, p. 219.

governing law of railroad practice. To the extent, that the monopoly inherent in the railroad franchise was a menace, it is of the utmost importance to recognize that current railroad regulation safely controls it."

* * * * * * *

"In so far as government policies have been designed, by Federal intervention, to create and maintain competition with or among railroads as a defense against monopoly, they should be abandoned as wasteful and unnecessary. Regulation is sufficient."

The Public Necessity

In view of the foregoing, an attempt to appraise the fundamentals of the situation may be in order. The public necessity requires a satisfactory transportation service. A satisfactory service is one that is, to the highest practicable degree, adequate, efficient, and economical. Although these characteristics are more or less interdependent, they may be commented upon separately.

There seems to be general agreement that the railroad plant is adequate for the transportation needs it is called upon to meet and that it will continue to be so for some time to come. The total revenue freight transported by Class I rail carriers of the United States in 1929 and 1935, is shown in Table 6, Columns (2) and (3). The transportation performance of the major rivers of the Mississippi Basin—the Ohio, Missouri, and Mississippi—covering river traffic between Pittsburgh, Pa., Minneapolis, Minn., Sioux City, Iowa, and New Orleans, in 1935 was as shown for comparison in Column (4), Table 6; the figures for waterways include materials used in river improvement work as well as materials rafted.

TABLE 6.—Comparison of Revenue Freight Transported

Revenue freight transported:	1929	1935	
	Railways	Railways	Waterways
(1)	(2)	(3)	(4)
Millions of tons	2 452	1 427	41
Millions of ton-miles	447 322	282 037	5 487

In a broad sense, the railroads serving the Valley have a reserve capacity at least proportionately equal to that of the railroads of the United States. Considering that the railroad plant was not overtaxed in 1929, and that its productive efficiency has been increasing and will continue to increase, it would appear that the construction, maintenance, and operation of waterways in the Valley is no longer justified upon the broad ground that other agencies of transportation are, or will be, inadequate in capacity to serve the public need.

From the viewpoint of efficiency, the rail carriers have certain inherent or natural advantages, such as a superior universality and flexibility of service, and the power to render continuous transportation without delay or expense for transfer at intermediate points. Under an equality of conditions the relative efficiency of two or more types of transportation service will be clearly revealed by the test of public use. These conditions are such as would cause each type to offer a service reflecting its own natural advantages or disadvantages, as well

as any superior or inferior degree of management, and are more or less as follows: (1) Substantial equality in respect of beneficial, restrictive, or punitive legislation and regulation; and (2) a system of service charges covering the total cost of the service.

Under such conditions there is little doubt as to the direction in which the verdict of public use would point. It is well known, however, that such equality of conditions does not now exist. The rail carriers are subject to a great mass of restrictive legislation and regulation, originally purporting to be in the public interest and now considered by many to extend far into the field of management. On the other hand, the river carriers are largely free to pursue such policies and practices as may seem to them appropriate, and are further enabled, by acts of Government, to offer their service for a charge which does not cover its total cost by a substantial margin; in other words, the users of the service are paying a part of the cost and the taxpayers are paying the remainder.

Even under the present inequality of conditions, the superior efficiency of rail transportation, based both upon its natural advantages and also upon alert and progressive management within the limits imposed by regulation, is generally conceded. Definite expression of this judgment may be found in voluminous shipper testimony before the Interstate Commerce Commission in one of its current general proceedings.[8]

Relative Costs

The third major requisite of transportation is that it be economical. Economy is necessarily relative, and certain approximately known elements of cost of river and rail transportation may properly be compared.

Broadly, the total cost of rail transportation to the public may be measured by the charge for the service, the freight revenue. In 1935 the average charge, or revenue, per ton-mile of revenue freight transported by the Class I rail carriers of the United States was 9.9 mills, of which 0.7 mill was directly refunded to the public in the form of taxes, thus reducing the average total charge to 9.2 mills per tone-mile. Substantially these figures apply to the railroads serving the Valley. For convenience, the rounded unit of 1% per ton-mile may be kept in mind as the outside total cost to the public of rail transportation.

The rivers of the Valley are of that class of waterways requiring large expenditures for navigation, and these expenditures do not appear in the accounts of the river carriers. Neither the costs of these carriers nor their charges for service are a measure of the total cost to the public of transportation on these rivers. This total cost includes at least the following elements: (1) Expenditures upon river improvement for purposes of navigation (*a*) by the United States; (*b*) by States and political subdivisions; and (*c*) incidental costs to public and private interests; (2) public expenditure on river terminals; and (3) service charges or costs of river carriers.

Expenditures by the Federal Government

The portion of river transportation cost paid by the Federal taxpayers, when reduced to the ton-mile unit, differs widely as between the rivers of the

[8] I. C. C. Docket No. 26 712, Rail and Barge Joint Rates.

Mississippi Basin. In the following paragraphs these values are given for the major rivers, the Ohio, the Upper Mississippi, the Missouri, and the Lower Mississippi, and for a few of the tributaries. The data are derived from the Annual Reports of the Chief of Engineers, U. S. Army, and are for the year 1935. which is the latest for which ton-mile data are available. Interest at $3\frac{1}{2}\%$ and current maintenance costs are used, and no allowance is made for amortization. To produce unit costs comparable with those of rail carriage, a circuity factor is used, representing the excess of river distance over rail distance.

In order that the discussion may include every possible factor favorable to river transportation, a number of findings are quoted from the report of the Mississippi Valley Committee of the Public Works Administration, which seems to be the latest comprehensive and independent study of the subject. This Committee, stated by the Administration to be "a group of the nation's leading scientists and technicians," devoted a year to the preparation of a report upon the use and control of water in the Mississippi Basin, which was made public in December, 1934. The membership of the Committee included the Chief of Engineers, U. S. Army, and its attitude was one wholly sympathetic with river values. In considering the following data it should be remembered that transportation on these rivers is no new or untried experiment; it is older than rail transportation—as old as commerce in the Valley.

Ohio River.—During the period of the Revolution considerable trade developed between Ohio River settlements and New Orleans and *via* New Orleans to the Atlantic seaboard. A regular packet service of keel-boats was established on the river in 1794. The course of the river is largely parallel to the main currents of traffic; it flows through a section highly industrialized for many years and in which bulk commodities are found in great quantities. Tonnage on the Ohio exceeds that on the Mississippi from Minneapolis to New Orleans.

Notwithstanding these favorable factors, Federal expenditures on the Ohio River amount to 5.5 mills per ton-mile of river traffic. To make the total cost as low as the total for rail transportation would require that all the other costs of river transportation previously mentioned aggregate not more than 3.7 mills per ton-mile. It seems improbable that such is the case.

The Monongahela River should be mentioned as a case exceptionally favorable to low-cost river carriage. Its tonnage, but not its ton-mileage, has been larger than that of the Ohio; in 1935 it was about 86% of the latter. Disregarding circuity, which is slight, the Federal contribution to cost of transportation on this river is 1.3 mills per ton-mile; 85% of the tonnage is coal, moved down stream from mines along the river.

If it be asked, what interests have benefited from the expenditure of about $185 000 000 of Federal money on the Ohio alone, the Mississippi Valley Committee of the U. S. Public Works Administration answers:

"Private carriers—largely coal and steel companies—now transport more than 95% of the total commerce on the Ohio River."

* * * * * * * *

"Practically all of this traffic is handled by private carriers who pay no tolls. In other words, under present policy, the construction costs and the operation and maintenance costs of navigation projects are at Federal expense, and no

part of either of these costs is directly returned to the Government. The Committee believes that this policy should be modified."

Mississippi River Above the Ohio.—The first steamboat is said to have ascended to Fort Snelling, near St. Paul. Minn., in 1813. During the era of active settlement in the Northwest, beginning about 1845, the river carried a considerable commerce. Following the extension of railroads into the Trans-Mississippi region, traffic left the river rapidly because of its natural disabilities, such as the condition of the channel, its closure during the winter, and chiefly its location across the direction of traffic.

The contribution of the Federal taxpayers to the cost of river transportation on the main stem between Minneapolis and the mouth of the Ohio is 2 cts per ton-mile and maintenance alone amounts to 0.6 ct per ton-mile. One-quarter of the total tonnage is material for river improvement.

The Committee finds that as of June, 1934, expenditures on the Upper Mississippi System, plus expenditures required to complete the present project, exceed $217 000 000, exclusive of $40 000 000 for maintenance and operation; the Committee remarks:

"It is not possible by any calculations of business accounting to discover an economic justification for the vast expenditures on the projected improvement of these waterways; especially from the prevailing viewpoint of self-liquidation, but also even from the viewpoint of complete coverage of cost of maintenance and operation."

Missouri River.—Although the first steamboat ascended the Missouri River in 1819, its commerce has always been relatively insignificant. Its natural channel conditions are particularly unfavorable for navigation and its location up stream from Kansas City is equally unfavorable for the needs of traffic.

Disregarding circuity and any expenditure on the Fort Peck project, the cost to the Federal taxpayers of river transportation between Sioux City and the mouth is 22 cts per ton-mile of total tonnage and the maintenance cost alone is 4 cts per ton-mile. In 1929, more than 99% of the total tonnage was material for river improvement work. In later years, the percentage is simply stated as "large"; however, in 1935, rafted piling, stone, and sand made up 84% of the total tonnage.

The Committee finds that the total construction cost of improvements under way will be about $250 000 000 and characterizes this expenditure as having "very doubtful justification." Considering that $3\frac{1}{2}$% interest alone would require a ton-mileage of about forty-three times the present (in itself largely composed of Government materials), in order to reduce this interest cost to 1 ct per ton-mile, the characterization may be pronounced conservative.

As to the possibility that Missouri River traffic will ever be such as to justify this expenditure, or any substantial part of it, the Committee comments:

"Assuming that the more optimistic estimates of future traffic on the river will be realized, the savings to shippers which would result from free operation of the waterway would exceed by only a small margin the annual charge to be borne by the public for maintenance and interest on the investment. *** Most current estimates of prospective tonnage are exceedingly liberal, and if the present traffic on the section below Kansas City may be accepted as in any

degree indicative of future prospects, the probability of those estimates ever being realized, even on a free waterway, is very slight."

To complete the picture, it should be mentioned that the construction period of the project has produced considerable business for the constructors, the producers of materials, and the transportation agencies, including the rail carriers.

As to the tributaries of the Missouri, the Committee finds that for the 5-yr period, 1928–1932, excluding non-recurring tonnage for construction of the Bagnell Dam, river traffic on the Osage cost the Government \$3.50 per ton-mile for maintenance alone.

Mississippi River, Cairo, Ill., to New Orleans, La.—On the Mississippi below its three great branches one would expect to find, if at all, the great national waterway, carrying a large volume of traffic at a cost less than the possible range of any transportation cost by land. What are the facts?

Up to June 30, 1935, the total cost of construction and maintenance work done by the Mississippi River Commission above the Head of Passes was about \$417 000 000, of which a relatively small amount pertains to the river above Cairo, Ill. The 1935 maintenance was \$5 177 000. In recent years these figures include expenditures for flood control which are not segregated from expenditures for navigation. However, the Committee finds that the cost of improving this section of the river (Cairo to New Orleans) for navigation, including the amount appropriated for this purpose under the Flood Control Act of 1928, may be estimated conservatively as \$190 000 000 and that the average annual maintenance is at least \$2 700 000.

The Committee also finds that the average annual traffic for the 5-yr period, 1928–1932, exclusive of material used in Government work and of the coastwise and foreign traffic below Baton Rouge, La., was about 1 681 000 000 ton-miles and that, after allowing for circuity, the total Government subsidy provided to shippers by river would be about 9 mills per ton-mile.

Disregarding any expenditures subsequent to those considered by the Committee, a similar calculation for the ton-mileage of 1935 shows a subsidy of 7.2 mills per ton-mile.

The addition of costs pertaining to river terminals and the costs or service charges of river carriers will obviously produce a total ton-mile cost on this section of the river greater than that of rail haul.

The Committee comments that "the matter seems to merit more thorough and comprehensive consideration than it previously has received."

Tributaries of the Lower Mississippi.—These tributaries present a picture unfavorable to the economy of river transportation. Table 7 shows, for 1935, on the more important tributaries, the dominant tonnage, the total Federal subsidy per ton-mile, and that part of the subsidy represented by the year's maintenance cost. Circuity is disregarded.

Major Rivers Consolidated.—More broadly comparable with the substantially uniform unit cost of rail transportation, is the consolidated Federal subsidy to transportation on the aforementioned four major rivers. For the year 1935, the annual charge pertaining to Government expenditures on these rivers is

TABLE 7.—DOMINANT TONNAGE AND FEDERAL SUBSIDIES, ON TRIBUTARIES OF THE LOWER MISSISSIPPI RIVER

River	Character of tonnage	Total subsidy, in cents per ton-mile	Required for maintenance only, in cents per ton-mile
St. Francis	96% logs and piling rafted	1.7	1.3
White	98% sand, gravel, and logs	2.3	1.8
Arkansas	99% sand, gravel, and logs	9.6	4.7
Red	60% logs	45.7	7.6
Ouachita and Black	36% logs, gravel, and riprap	2.8	1.7
Yazoo and Big Sunflower	45% sand, gravel, and logs	13.3	3.4

$29 000 000 or, allowing for circuity, 10.1 mills per ton-mile of commercial river traffic.

OTHER EXPENDITURES

From time to time States and agencies of the States have made considerable expenditures on improvement of waterways in the Valley. In the case of highway and railroad bridges over these waterways, the additional expenditures necessary to provide the type of structure and the horizontal and vertical clearances imposed in the interest of navigation have been considerable, and there is a continuing element of cost in lifting traffic to the elevation of many fixed bridges. A number of cities in the Mississippi Valley have provided terminals, at their own cost, for the use of river carriers, and the results of these expenditures have not been satisfactory to the municipal taxpayers in all cases.

At present, there appears to be no comprehensive and authoritative compilation of these other expenditures on any of the major waterways of the Valley. Although the total would doubtless increase, perceptibly, the unit cost of river transportation derived from the cost of channel improvement, it is probably small in comparison with Federal expenditures.

REVENUE OF WATER CARRIERS

The Inland Waterways Corporation, owned and controlled by the United States, is the dominant common carrier by water in the Valley. Operating the Federal Barge Lines with modern and efficient equipment, its freight revenue is presumably an approximate measure of that part of the cost of river transportation represented by the necessary charge for carrier service. In 1935, this freight revenue was 3.85 mills per ton-mile.

It appears, therefore, that 3.5 or perhaps 4 mills per ton-mile should be added to the costs previously mentioned as representing the necessary revenue of the common carrier by river. The costs of private and contract carriers are doubtless less, although the cost of private terminals should be included; but, under present conditions, the provision at public expense of a free waterway to these carriers seems open to serious question.

Again, the circuity factor should be applied to the ton-mile cost or revenue of the river carrier, in order to make it comparable with the cost of rail transportation. On the Lower Mississippi, for example, the two factors—Federal

expenditures and common carrier revenue—produce a ton-mile cost to the public of 12 or 13 mills as against a total cost of 9 or 10 mills for rail carriage.

Any consideration of the feasibility of common-carrier traffic on the river by the use of private capital should include the experience of barge lines operating between St. Louis, Mo., and New Orleans, from 1875 and 1903, as given by Professor Dixon.

The equipment appears to have been up to date for the time. The barges had a capacity of from 50 000 to 60 000 bushels of grain and the lines carried general merchandise as well. Fuel was carried for the round trip and prompt and reliable schedules were planned. One shipment in 1880 included: 6 757 bbl of flour, meal, and grits; 6 710 sacks of corn, oats, and bran; 1 500 packages of meat and lard; 150 bales of hay; and 24 992 bushels of bulk corn. In 1887, four lines were operating 16 towboats and 120 barges and were later consolidated into one company.

For the seventeen years, 1887 to 1903, inclusive, the published rates on grain from St. Louis to Liverpool, England, by river *via* New Orleans, were from 5 to 9 cts per bushel lower than those *via* rail to New York, N. Y., and, in the latter year, were 6 cts lower. Notwithstanding this differential, the movement of bulk grain by river showed a general decline after 1890, and by 1903, the barge lines seem to have passed out of the picture as an unprofitable investment.

At present, grain is not an important part of river tonnage. In 1935, after eliminating the tonnage in foreign and coastwise commerce on the lower river, wheat, corn, rice, and all other grains form the following percentages of the total tonnage: Ohio, 0.05; Upper Mississippi, 4; Missouri, 0.8; and Lower Mississippi, 1.6.

Intangibles

The suggestion is sometimes made, and at first glance appears pertinent, that consideration of expenditures that have already been made on the waterways is "academic," apparently using the term in a somewhat disparaging sense. In effect, however, this suggestion is the same as stating that present and future action should ignore the experience of the past.

It is also suggested, at first with apparent force, that although the existing investment in waterways may not have been wholly wise, this investment should be preserved or "salvaged" by providing for its continued use. It is generally admitted that the United States has a very great surplus of transportation facilities. In so far as transportation efficiency is in the public interest, the use of, and the addition to, the less efficient agency is a drain upon the nation's resources and a check upon its total productivity. The real present question is not as to the salvage of an investment; it is as to the preference for the smaller and less efficiently functioning investment over the larger and more efficient.

No agency of transportation, rail, water, or any other, regardless of investment, should be preserved and fostered simply because of its past performance. If and when it becomes relatively inefficient, outmoded, and obsolete, the public interest requires that it be retired from use as rapidly as may be practicable. From the viewpoint of past performance, however, whatever it may be con-

sidered to be worth, there is something to be said for the rail carriers. In the words of the Mississippi Valley Committee:

"A nation with our territorial extent, population, culture, and vital dependence of one section upon another, is unique. It has been made possible only by an elaborate and adequately supported railroad system. Therefore, any move, however desirable it may be in itself, which unduly infringes upon the serviceability of railroads, is to be deprecated."

Summary

Considerations of "romance and tradition," the emotional viewpoint, although based upon an essential part in the national life formerly played by the waterways of the Mississippi Valley, is no longer a sound basis for expenditures upon the improvement and maintenance of these waterways for transportation purposes. The justification of these expenditures in the public interest has been based upon the supposed necessity of transportation facilities additional and supplementary to an inadequate capacity of the rail carriers, and also upon the assumed necessity of improved rivers as competitors of the railroads and regulators of their rates. These considerations may or may not have been valid in the past. To-day, they are no longer relevant; the public interest is based upon the relative economy of such available agencies of transportation as are capable of rendering a satisfactory service.

The total unit cost of river transportation in the Valley, including that part of the cost paid by the taxpayers, is, in general, greater than the total unit cost of rail transportation. Due to the fixity of river location, any benefits of river improvement paid for from the National Treasury tend to accrue, not to the nation as a whole, but to special or localized industries and localities.

The nation's credit is of the highest and is expected to remain so. Consequently, public expenditures will have to be repaid, with interest, by taxation. Expenditures which are an exercise of a power of Government, rather than an exercise of one of its basic and essential functions, should rest upon a basis of economic justification. Otherwise, they constitute a levy upon the nation's total consumption of goods and services.

Conclusions

The application of the foregoing to the matter of waterway improvement in the Valley would appear to prompt some such suggestions as the following:

(1) Projects upon which the cost of Federal maintenance exceeds the cost of using other transportation facilities, should, if such other facilities are available and adequate, be retired forthwith, or be turned over to such local interests as may be able to operate and maintain them on an economic basis.

(2) Projects upon which the cost of Federal maintenance is less than the cost of using other available facilities should, in justice to the taxpayers who provided and now support them, carry such service charges to the users as may be equitable in each individual case.

(3) Additional expenditures should not be made except upon a basis of economic justification; a definite showing, after consideration of all costs

involved and a realistic appraisal of traffic to be expected, that a satisfactory transportation service will be produced at a total cost less than that of existing agencies.

In the words of the Mississippi Valley Committee:

"It is believed that this phase of the subject has in some instances in the past been too lightly considered—if considered at all. Economic justification should be definitely established early in the study of any proposed river or canal improvement."

AMERICAN SOCIETY OF CIVIL ENGINEERS

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PAPERS

MEASUREMENT OF DÉBRIS-LADEN STREAM FLOW WITH CRITICAL-DEPTH FLUMES

By H. G. Wilm,[1] Esq., John S. Cotton,[2] Esq., and H. C. Storey,[3] Esq.

Synopsis

Heavy burdens of erosion débris in streams of the San Dimas Experimental Forest, in California, have been found to cause substantial errors in measurements of discharge at some of the gaging stations. Stream-flow records are kept at a number of points throughout the Experimental Forest. Furthermore, there are two sets of triplicate water-sheds, each shed having an area somewhat less than 100 acres, immediately below each of which is located a Parshall flume, a V-notch weir, a small reservoir, and a flat-crested weir over the dam forming the reservoir. The flow from these water-sheds is first measured through either the Parshall flumes or V-notch weirs, according to the volume of flow. Its débris content can then be measured in the reservoir, and the clear water can be measured as it passes over the flat-crested weir of the dam.

However, there are numerous other points on the water-sheds below which the entire dependence is placed on the Parshall flumes or V-notch weirs for the determination of stream flow. Due to the presence of large quantities of eroded material in all heavy flows, the results at these points have been somewhat erratic. This was due largely to deposition of débris on the level approach to the throats of the Parshall flumes during the falling stages of the stream flow, and to the filling of the weir boxes with silt which soon put the V-notch weirs "out of commission."

The experiments described in this paper were conducted with the consultation and under the supervision of E. W. Kramer, M. Am. Soc. C. E., as Regional Engineer (California Region, United States Forest Service) now (1937) Regional Director (Federal Power Commission, San Francisco, Calif.) and under the authority of Professor E. I. Kotok, Director, and C. J. Kraebel,

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Project Leader, of the California Forest and Range Experiment Station (United States Forest Service). The objective was to improve the present gaging stations so that they would yield reasonably accurate results with bed-load material present in the stream measured. Included in the study were also tests of the Parshall flume as constructed in the Experimental Forest, to check its accuracy with clear and loaded flows.

Introduction

The California Forest and Range Experiment Station has instituted an intensive program of investigations for the purpose of solving problems concerning the inter-relationship of mountain vegetation and water, with the objective of developing methods of water-shed management which should provide a maximum yield of usable water with a minimum of soil erosion. These studies are largely concentrated in the San Dimas Experimental Forest, a chaparral-

Fig. 1.—Large Gaging Station, San Dimas Experimental Forest; Parshall Flume and 90-Degree V-Notch Weir

covered mountain area of 17 000 acres in the San Gabriel Mountains, approximately 30 miles east of Los Angeles, Calif.

In connection with studies of other phases of the problems, analyses have been made of rainfall-run-off ratios in seventeen drainage basins of various areas, from 0.06 sq mile to 14.1 sq miles.

Rainfall averages for these water-sheds, obtained by means of a system of nearly 400 standard rain gages and 15 intensity gages, are known to be accurate within 5 to 10% for the large areas, and from 3 to 5% for the small, intensively studied water-sheds. To date, however, stream-flow records have been of greatly inferior accuracy, due to the presence of quantities of eroded material in all storm peak flows. Deposition of this material results in erratic and undependable volume measurements in the gaging stations.

Sixteen of the seventeen gaging stations completed at present (1937) were constructed during the course of a single dry season from designs based on the best existing practice. (To the writers' knowledge, no control flume or weir had been designed previous to this time, which would operate as well as the Parshall flume in measuring silt-laden flows.) In each case a Parshall flume,[4] of throat width (3 ft to 30 ft), sufficient to handle the largest probable flows, was built to operate in conjunction with a 90° V-notch weir, intended to measure small continuous flows. Measurement of water stages in each is made by a continuous time-stage recorder.

In the design of Parshall flumes for the Experimental Forest, several minor modifications had been introduced, in order to increase their capacity and improve their efficiency in measuring silt-laden flows. The rising floor section down stream from the throat (see Fig. 1) was replaced by a downward sloping floor, the converging sections were lengthened, and the walls were increased in height.

Storm flows during the first rainy season following construction soon demonstrated the inadequacy of this type of control in measuring flows containing a substantial bed load. Even small post-storm streams carried enough sand and gravel to fill weir boxes quickly, and to build up a deposit on the flume floors so that they no longer functioned as critical depth meters. It is believed that the flumes should still measure large flows with reasonable accuracy, as the effect of bed load would constitute a much smaller percentage of the total flow; but as the stations stand at present they fail to give a satisfactory measurement of small and intermediate débris-laden flows, which form the run-off resulting from most storms.

Below the gaging stations, at each of the smallest, intensively studied watersheds, is a cement-lined débris reservoir. The water depths measured in this basin and over the broad-crested weir spillway afford an efficient check on stream-flow measurement, in some cases furnishing the only reliable data on flow from the water-sheds studied.

Purpose of the Investigation

At the request of the Regional Engineer, the experiments described in this paper were set up in March, 1935, with two major objectives: (1) To test the effect of construction modifications on the rating of Parshall flumes in the San Dimas area; and, (2) to adapt the present stations to measurement of flows of all sizes, either by improving the débris-transporting characteristics of the Parshall flume, or, failing this, by attempting to develop a control flume that would function satisfactorily under a wide range of conditions and might be used in conjunction with the present stations to measure flows of intermediate volume.

Experimental Equipment

Through the courtesy of the San Dimas Water Company and the Los Angeles County Flood Control District, water from the San Dimas (flood-

[4] "The Improved Venturi Flume," by R. L. Parshall, Assoc. M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 89 (1926), p. 841.

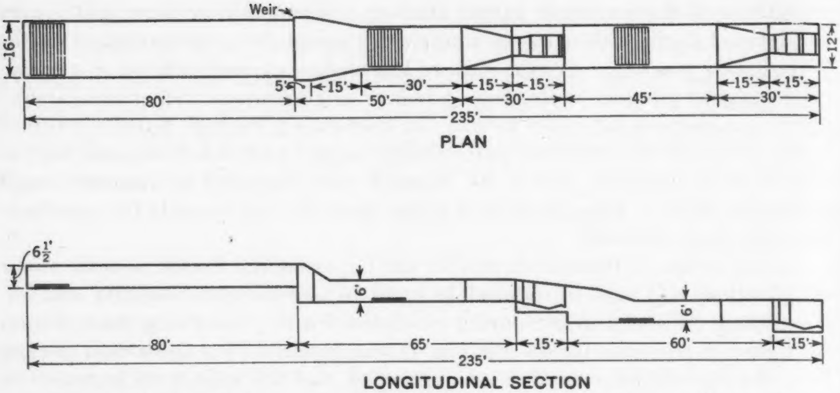

FIG. 2.—PLAN AND SECTION OF SAN DIMAS EXPERIMENTAL FLUME, SHOWING PORTION OF SHARP-EDGED CONTROL WEIR, EXPERIMENTAL FLUMES, AND RAFT BAFFLES

control) Reservoir was made available for experimental purposes, in any quantities desired up to 250 cu ft per sec.

Below the outlet gates in this dam was constructed a water-tight timber flume, lined with tar-paper, 16 ft wide, 6.5 ft deep, and 80 ft long, beyond the end of a short concrete channel leading from the sluice-gates. At the end of this section of flume was installed a 16-ft sharp-edged, steel suppressed weir, set level with a maximum variation of $\pm$ 0.001 ft. By the installation of additional bulkheads and short vertical sharp edges, this weir could be shortened to form a contracted weir of any desired length.

Beyond the weir the structure was continued, narrowing to a width of 12 ft, for 155 ft. At the weir, the floor of this section is 4.5 ft below that of the

FIG. 3.—GENERAL VIEW OF FLUME AS SEEN FROM TOP OF SAN DIMAS DAM

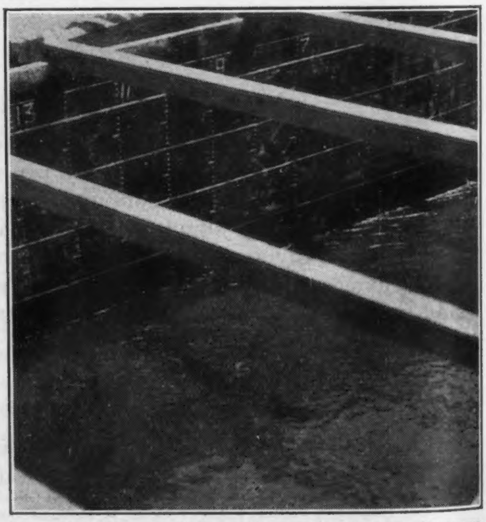

FIG. 4.—WALL OF 5-FOOT PARSHALL FLUME, SHOWING ARRANGEMENT OF METAL STRIPS USED IN PLOTTING WATER SURFACE CURVES

weir channel; and a second drop of 6.8 ft occurs in the middle of the lower structure. Experimental flumes were installed here and at the down-stream end of the structure. Figs. 2 and 3 show the design and a general view of the entire installation.

Water depths over the weir and at both experimental controls were measured in 1-ft iron stilling-wells, by means of milliammeter point gages designed by the Staff of the Experiment Station, and accurate within ± 0.001 ft. In addition, water-surface curves were plotted by the use of metal strips tacked to the walls of the flumes being tested, at horizontal intervals of 1 ft and at vertical intervals of 0.1 ft (see Fig. 4).

Control Flumes Studied

In every test made, at least two flumes were rated simultaneously at the two stations constructed in the large test flume. Tests made with various experimental structures in the San Dimas test flume, listed in chronological order, and noting the length of weir used as a control for each test, are as follows:

I.—Tests of Modified Parshall Flume.—

(*A*) Five-foot flumes (rating with clear water), with 16-ft suppressed weir as control, using flows from 14.08 cu ft per sec to 234.5 cu ft per sec: (1) Same design as that adopted in the Experimental Forest gaging stations, with level floor in converging section; and (2) same design as that adopted in Experimental Forest gaging stations, except that the floor of the converging section was given a 5% slope.

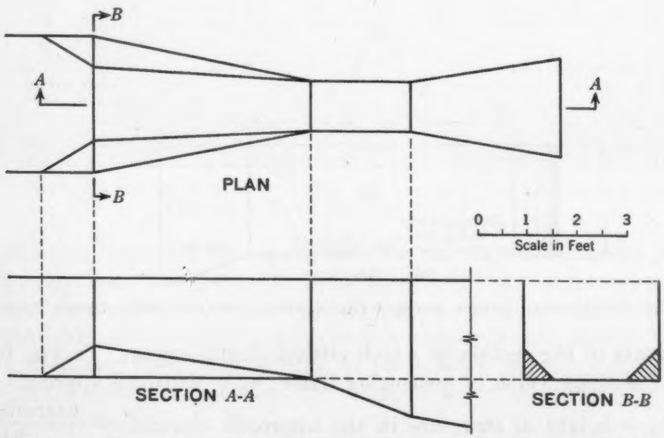

FIG. 5.—PARSHALL DESIGN, 45-DEGREE FILLETS IN A 1-FOOT PARSHALL FLUME

(*B*) One-foot flumes with 3-ft contracted weir as control, using flows from 0.11 cu ft per sec to 16.0 cu ft per sec: (1) Rating with clear water: (*a*) Level floor in converging section; 45° wing-walls extended from up-stream end of flume to walls of 12-ft channel; (*b*) floor of converging section on 5% slope; (*c*) floor of converging section on 3% slope; (*d*) level approach floor, with fillets

installed on this floor from the throat back to the up-stream end of the flume, and set at an angle of 45° from the floor to the converging side walls (Section *B–B* Fig. 5). This fillet design was suggested by Mr. Parshall[5] for the purpose of improving the efficiency with silt-laden flows; and (*e*) level floor in converging section, wing-walls extended parallel to each other from the up-stream end of the flume for 10 ft, instead of diverging as 45° wing-walls. (2) Rating with débris-laden flows: Same installations as in Item *B*–1.

(*C*) Six-inch flume, with a 3-ft contracted weir as control, using flows from 0.20 cu ft per sec to 2.71 cu ft per sec. The purpose of this test was to ascertain the effect on the rating formula of entrance conditions radically different from standard. This flume was installed in the side of the main structure below the lower 1-ft flume, and with its axis at right angles to that of the main structure. The result was considerable turbulence and an abrupt change in flow direction.

(*D*) Three-foot flumes with an 8-ft contracted weir as control, using flows from 0.62 cu ft per sec to 53.52 cu ft per sec: (1) Level floor, but with a series of low fillets along the floor. These fillets were triangular in cross-section, tapering in height from 4 in. at the up-stream end of the flume, to 1 in. at a point 1 ft up stream from the flume throat. Sloping ramps were constructed on the ends of each fillet.

II.—Experiments with Other Critical-Depth Flumes.—In the course of the studies, a new type of measuring flume was developed, which functions as a broad-crested weir in which water depths are measured at a point in the flume

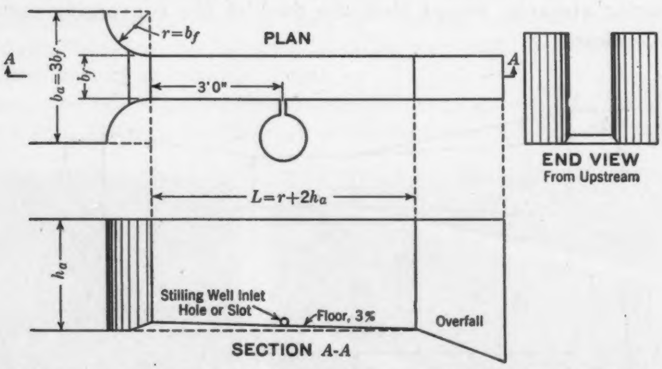

FIG. 6.—GENERAL DESIGN FOR SAN DIMAS RECTANGULAR CONTROL-DEPTH FLUME

down stream of the section in which critical depths occur. In Fig. 6, for example, b_f denotes width of measuring flume; b_a = width of approach channel $= 3b_f$; h_a = height of structure in the approach channel $= \dfrac{\text{maximum flow}}{5\,b_f}$; L = length of the measuring flume $= r + 2h_a$; and r = radius of the transition $= b_f$. The object of this design is to provide for measurement of flow at critical or super-critical velocities, sufficient to transport débris rapidly through the flume and to keep its floor scoured clean. This flume differs in function, design,

[5] Memorandum concerning the effect of fillets placed against the walls of the converging section of a 1-ft Parshall measuring flume, by Ralph L. Parshall, U. S. Bureau of Agricultural Engineering, Fort Collins, Colo., March, 1935.

and point of depth measurement from the Venturi flume described by H. K. Palmer and F. D. Bowlus, Members, Am. Soc. C. E.[6] The meter described by Messrs. Palmer and Bowlus would not function satisfactorily for measuring flows containing bed-load, as measurements made up stream of the "critical" section must necessarily be affected by changes in approach velocity, caused by deposition of transported material in the back-water zone above the flume's entrance transition. Because of the necessity of measuring loaded flows, no conventional broad-crested weir could be used, as its contraction would soon become obliterated by accumulated débris. Therefore, a standard flume transition was constructed in the new flume to replace the bottom contraction.

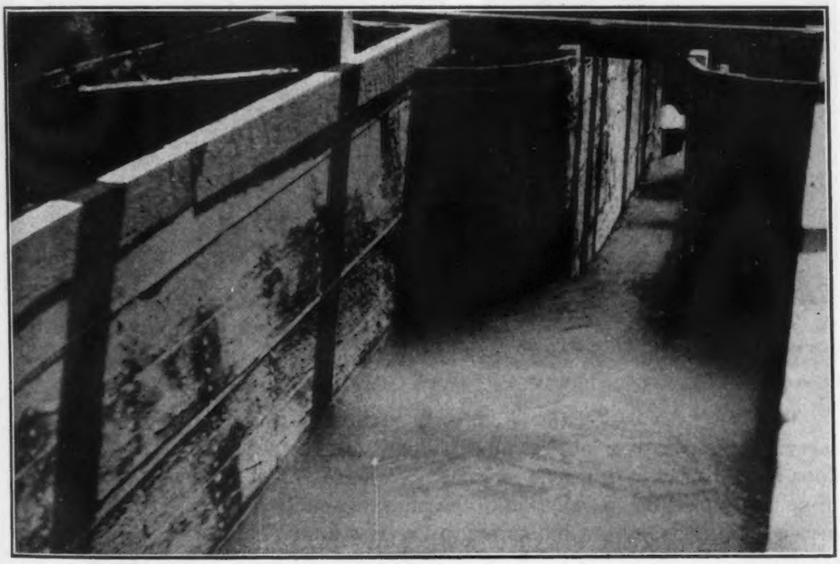

FIG. 7.—VIEW FACING DOWN STREAM; 1-FOOT RECTANGULAR, CRITICAL-DEPTH FLUME; FLOW, 2.71 CUBIC FEET PER SECOND

The transition is the cylinder quadrant (see Fig. 7), described[7] by Fred C. Scobey, M. Am. Soc. C. E., in 1933, with a radius of curvature equal to the width of the flume. This type of transition was adopted because of its simplicity and relatively short length, and because, according to Mr. Scobey's experiments, the head loss through it should, in most cases, be no greater than in more complex transitions, such as the reversed curve described[8] by Julian Hinds, M. Am. Soc. C. E., in 1928.

Below this transition the measuring flumes tested were either rectangular, as shown in Fig. 8, or trapezoidal in cross-section, with a floor laid on a 3% grade. The trapezoidal flumes had a wall slope of 1 on 0.25 (see Fig. 9), the object of

[6] *Transactions*, Am. Soc. C. E., Vol. 101 (1936), p. 1195.

[7] "The Flow of Water in Flumes," by Fred C. Scobey, *Technical Bulletin No. 393*, U. S. Dept. of Agriculture, December, 1933.

[8] "The Hydraulic Design of Flume and Siphon Transition," by Julian Hinds, *Transactions*, Am. Soc. C. E., Vol. 92 (1928), pp. 1435–1439.

this cross-section having been to increase capacity while retaining accuracy for low flows, and to attempt to hold critical depths at points close to the "crest," or to the down-stream end of the entrance transition.

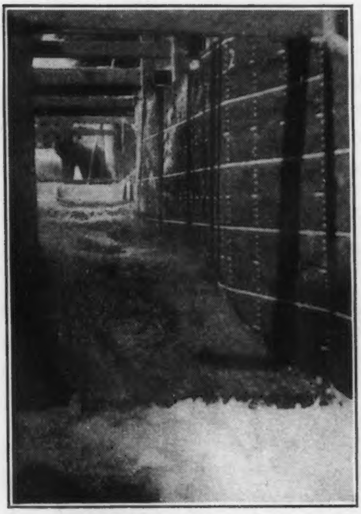

FIG. 8.—THREE-FOOT RECTANGULAR CRITICAL-DEPTH FLUME; VIEW FACING UP STREAM; FLOW, 12.68 CUBIC FEET PER SECOND

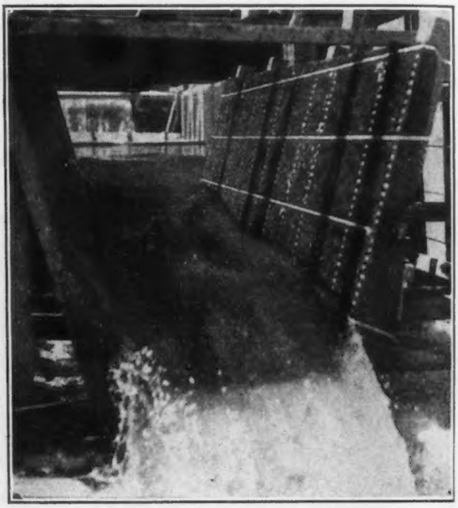

FIG. 9.—TWO-FOOT TRAPEZOIDAL CRITICAL-DEPTH FLUME; FLOW, 22.97 CUBIC FEET PER SECOND

The up-stream end of the 3% floor in each flume is formed by a short ramp, carrying over on to a level approach floor. In the test flumes, the approach floor was covered with gravel at stream grade (about 3 per cent). Up stream from the transition, parallel or diverging wing-walls are carried back a short distance into the stream banks or, in the test flume, to the walls of the main structure.

A 3% grade was chosen for the floor slope within the measuring flume so as to insure maintenance of stable flow conditions, well below critical depths, even for shallow flows of water containing sufficient débris to raise the coefficient of hydraulic friction of the flume as high as $n = 0.025$. Although this allowance may seem liberal, it was considered necessary because of present inadequate knowledge of the quantitative influences of transported material on flow characteristics in flumes. The only disadvantageous effect of such a floor slope lies in its increasing velocities which tend to reduce the sensitivity of the instrument to small changes in flow. This effect may be minimized, however, by locating the stilling-well inlet reasonably close to the section in which critical depths occur. Furthermore, any such reduction in depth variation per unit of volume variation is largely compensated in practice by the facts that any error in measurement of head causes only approximately 1.3 times that error in computed volume; and that velocity of approach has no effect, theoretically, upon the flume's rating.

Preliminary ratings have been made for three sizes of flume in both the rectangular and trapezoidal designs: 1, 2, and 3-ft widths in the former, and

0.5, 1, and 2-ft bottom widths in the latter. The 1-ft flumes were rated by 3-ft and 8-ft weirs; the 8-ft weir was used also for the larger widths of the flume. A 1-ft rectangular flume with a level floor was also tested, in order to verify the similarity in discharge behavior between this type of control flume and a broad-crested weir.

Experimental Procedure

In each test run, gate-openings in the dam were controlled by a staff member of the Los Angeles County Flood Control District, who was in continuous communication with a co-ordinator at the test-flume structure. To start the run, the gate was opened enough to fill all the basins in the test-flume structure, and then lowered to a small fixed opening. When the flow had reached stability, as indicated by constant point-gage readings at the lowest experimental flume, a signal was given and six to ten depth readings were made by separate observers at each point-gage, noting times of reading to the nearest $\frac{1}{4}$ min. Other observers plotted the water-surface curves in each flume being rated, and recorded the water depths in the channels up stream. Staff gage readings were also taken at the weir as a check on the point-gage, and current-meter velocities for the larger flows were measured to check the weir rating. After the completion of point-gage readings for each stabilized flow, the gate-opening was increased and the entire operation repeated until enough different volumes had been measured to insure the procurement of reliable rating curves. For the 6-in. Parshall flume only seven points could be taken; but for each of the others ten to fifteen separate volumes were measured.

Complete stability of flow through the gate was insured by the large head of water available behind the dam. Formulas used for the control weirs were:

For the 16-ft weir (Bazin's formula):

$$Q = \left(0.405 + \frac{0.00984}{H}\right)\left(1 + 0.55\left[\frac{H}{Z + H}\right]^2\right) L\, H\, \sqrt{2gH} \;\ldots\ldots (1)$$

and, for 8-ft and 3-ft contracted weirs:

$$Q = 3.33\,(L - 0.2H)\, H^{1.5} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (2)$$

in which Z = the height of the crest above the bottom of the channel. Although no volumetric or other completely reliable rating could be made of these weirs, their accuracy is believed to be well within limits required for the study ($\pm$ 2% to 3%), as demonstrated by their close correspondence with volumes computed from current-meter measurements, and by the agreement of observed discharges with points determined by formula.

In testing the behavior of débris-laden flows in the small Parshall and rectangular San Dimas flumes, a stream-bed mixture of gravel and sand was dumped as rapidly and continuously as possible by six men with two wheelbarrows into the channels just up stream from the flumes tested. In each case, water depths in the flumes were measured first with a stabilized flow of clear water; then, with the same rate of flow, débris was added as rapidly as possible, and the depths were remeasured.

Later, in testing the trapezoidal San Dimas flumes, bed-load material was added from large bunkers constructed above the flume structure just up stream from each flume. It was found difficult, however, to feed the débris uniformly from the bunkers, because the material became packed.

In computing results, all point-gage readings for each stabilized flow were averaged and checked with the corresponding staff-gage reading or the water-surface curve, and rating curves and formulas were obtained from the resulting average points. Variation of individual readings from their average seldom exceeded 0.01 ft.

Computations were made of velocities and energy gradients within each flume, in order to determine energy losses attributable to the transition and to friction and turbulence within the flume. The results were not satisfactory, probably owing to centrifugal effects in curved flow. However, as nearly as could be determined, losses through the transition and flume appear to be small.

All calculations were made with a 50-in. slide-rule and an electric calculator. Computed discharges obtained by "least squares" were checked against a straight line drawn through logarithmically plotted, observed discharges.

Results

Rating curves and formulas were computed for the flumes tested, and compared with Parshall's ratings of his flumes. The significant results of the experiment are shown in Tables 1 to 7, and summarized as follows.

Modified Parshall Flumes with Level Floor in Converging Section.—

(*a*) The logarithms of all rating points fit a straight line closely, with deviations which exceed 5% only in the 6-in. flume.

(*b*) For a 6-in. flume,

$$Q = 2.13\, H_a^{1.47} \quad \ldots\ldots (3)$$

Increasing the length of the entrance section of the Parshall flume and depressing the floor of the outlet section have little effect, if any, on the discharge characteristics of the flume.

(*c*) Flow in Parshall flumes, at depths as great as twice those originally rated, conformed with no increased error to computed rating formulas.

Alterations of the Parshall Flume.—The rating with clear water may be described as follows:

(1) A 3% to 5% slope in the floor of the converging section resulted in shooting flow at low stages. As depths were increased a hydraulic jump passed through the flume. All larger quantities flowed at depths greater than the critical, giving smooth rating curves with discharges for all heads considerably higher than standard.

(2) Introduction of fillets of either design resulted in discharges somewhat in excess of standard ratings. In the 3-ft flume with multiple fillets, low flows occurred at shooting velocities, and at all stages a slight drop was perceptible at the up-stream end of the fillets.

Effect of Débris on the Altered Flumes.—Excellent transportation of bed-load resulted as long as shooting velocities occurred. At higher stages, however, with normally slow velocities, débris movement was greatly reduced. The

introduction of fillets somewhat improved the transportation of fine material, but failed to be effective for bed-load ranging above fine gravel in coarseness.

When bed-load was present in the flow, observed discharges were erratically in excess of normal values for the flumes at all stages tested (see Table 1).

TABLE 1.—Effect of Débris-Laden Flows Upon Rating of Measuring Flumes

Reading No.	Observed upstream head, H_a, in feet	Rate of Flow, Q, in Cubic Feet per Second			Percentage deviation	Reading No.	Observed upstream head, H_a, in feet	Rate of Flow, Q, in Cubic Feet per Second			Percentage deviation
		Observed*	Computed*	Deviation (Column (2) − Column (3))				Observed*	Computed*	Deviation (Column (2) − Column (3))	
	(1)	(2)	(3)	(4)	(5)		(1)	(2)	(3)	(4)	(5)
(a) One-Foot Parshall Flume, 3% Floor, in Converging Section						(c) Three-Foot Parshall Flume with Multiple Fillets					
10	0.646	2.75	2.45	+0.30	+12.2	15a	0.966	13.73	11.85	+1.88	+15.9
7	0.885	3.83	3.94	−0.11	− 2.8	14a	1.131	16.11	15.15	+0.96	+ 6.3
8	1.295	12.10	6.67	+5.43	+81	13a	1.388	21.33	20.75	+0.58	+ 2.8
9	1.456	9.76	7.83	+1.93	+25						
(b) One-Foot San Dimas Critical-Depth Flume, 3% Floor						(d) Three-Foot San Dimas Critical-Depth Flume, 3% Grade					
17	0.693	3.89	3.90	−0.01	− 0.3	15a	0.683	13.73	13.80	−0.07	− 0.5
19	1.341	9.37	9.37	0.00	0.0†	14a	0.780	16.11	16.32	−0.21	− 1.3
20	1.645	12.11	12.31	−0.20	− 1.6†	13a	0.968	21.33	21.41	−0.08	− 0.4

* Observed head is for flows containing bed-load material; observed discharge is based on control-weir measurements; and computed discharge is based on San Dimas ratings for clear water. † Channel above flume built up to 25% with bed-load material.

These findings support field observations made at Experimental Forest gaging stations, where it was found that bed-loads are transported through Parshall flumes only at high discharges. Low flows carrying loads cause deposits of

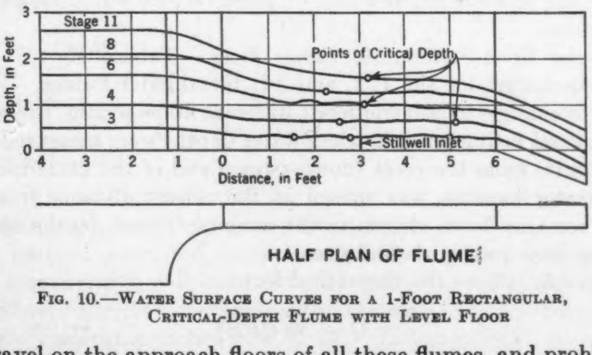

Fig. 10.—Water Surface Curves for a 1-Foot Rectangular, Critical-Depth Flume with Level Floor

sand and gravel on the approach floors of all these flumes, and prohibit the use of notch weirs by similar deposition. These facts result in serious errors and gaps in flow records.

High-Velocity Flumes (San Dimas Design).—Surface waves occurred in this flume (see Fig. 10), because a level floor does not supply gravitational energy to replace that lost in friction; therefore, the depths fluctuated at about the critical, within the flume. The resulting points lie on either side of a curve represented by the formula,

$$Q = \sqrt{g}\, H^{\frac{3}{2}} \qquad (4)$$

which is the theoretical formula for a broad-crested weir. The rating is unsatisfactory, as deviations of individual points from the curve are as great as 15% (see Fig. 11). Furthermore, addition of débris immediately caused the flow to jump above the critical depth down stream of the transition.

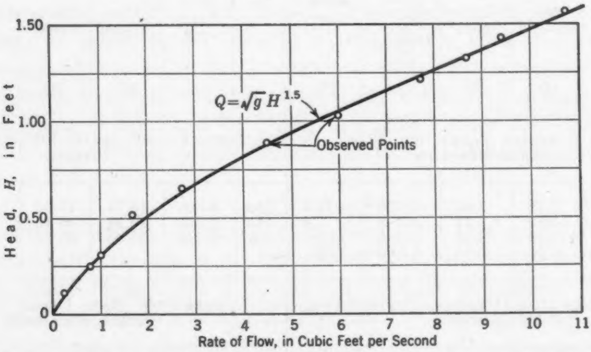

FIG. 11.—RATING CURVE FOR A 1-FOOT RECTANGULAR, CRITICAL-DEPTH FLUME WITH LEVEL FLOOR

As shown in Fig. 10, critical depths lay at points near the middle of the flume, where relatively parallel flow occurred. For the measurement of clear flows it might be feasible to use a flume similar to the San Dimas design, but with its floor set at a critical slope, so that parallel flow would be established at the critical depth and a rational formula might be applied. However, flow in such a flume would be very sensitive to small changes in floor friction or to the addition of silt or detritus, and any of these factors would result in unstable flow conditions.

Rectangular Cross-Section, 3% Flume Floor.—Table 2 shows observed and computed discharges for the 1, 2, and 3-ft rectangular flumes. With a small correction for changes in unit friction in different flume widths, these flumes gave the same general rating formula when water depths were measured at the same distance (3.0 ft) from the crest (down-stream end of the entrance transition). This piezometer location was chosen as the closest distance from the crest, which still remains down stream of the zone of critical depths at the highest water-stages measured in these flumes.

This formula follows the theoretical form,

$$Q = \sqrt{g}\, C B H^{\frac{3}{2}} \qquad (5)$$

in which C = an empirical constant to correct for the effects of floor slope and of energy losses in the flume. Based on present tentative ratings for flumes in

TABLE 2.—COMPARISON OF COMPUTED WITH OBSERVED DISCHARGES IN 1, 2, AND 3-FOOT RECTANGULAR SAN DIMAS FLUMES

Reading No.	Observed upstream head, H_a, in feet	Rate of Flow, Q, in Cubic Feet per Second			Percentage deviation
		Observed	Computed	Deviation (Column (2) − Column (3))	
	(1)	(2)	(3)	(4)	(5)
(a) ONE-FOOT FLUME: (EQUATION (9a))					
2	0.175	0.67	0.64	−0.03	−4.48
3	0.245	0.96	0.99	+0.03	+3.15
4	0.533	2.71	2.76	+0.05	+1.85
5	0.773	4.54	4.52	−0.02	−0.44
6	0.955	5.98	5.98	0.00	0.00
7	1.154	7.66	7.68	+0.02	+0.26
8	1.260	8.61	8.62	+0.01	+0.12
9	1.346	9.39	9.40	+0.01	+0.11
10	1.480	10.74	10.66	−0.08	−0.74
11	1.599	11.85	11.81	−0.04	−0.34
(b) TWO-FOOT FLUME: (EQUATION (9b))					
3	0.188	1.53	1.55	+0.02	+1.31
2	0.438	4.54	4.55	+0.01	+0.22
4	0.759	9.27	9.17	−0.10	+1.08
(b) TWO-FOOT FLUME: (EQUATION (9b)) (*Continued*)					
5	0.925	11.80	11.82	+0.02	+0.17
6	1.242	17.28	17.20	−0.08	−0.46
7	1.533	22.38	22.51	+0.13	+0.58
8	1.741	26.20	26.49	+0.29	+1.11
9	2.262	36.49	37.00	+0.51	+1.40
(c) THREE-FOOT FLUME: (EQUATION (9c))					
3	0.131	1.53	1.57	+0.04	+2.61
2	0.315	4.54	4.72	+0.18	+3.96
1	0.523	8.94	8.88	−0.06	−0.67
4	0.541	9.27	9.25	−0.02	−0.22
5	0.660	11.80	11.86	+0.06	+0.51
6	0.891	17.28	17.23	−0.05	−0.29
7	1.105	22.38	22.53	+0.15	+0.67
8	1.256	26.20	26.43	+0.23	+0.88
9	1.638	36.49	36.80	+0.31	+0.85
10	2.145	51.43	51.44	+0.01	+0.02

1-ft to 3-ft widths, the general formula is as follows:

$$Q = \sqrt{g}\left(\frac{1.120\, B^{0.040}}{H^{0.179} B^{0.320}}\right) B H^{\frac{3}{2}} \quad \ldots\ldots\ldots\ldots\ldots\ldots (6)$$

Supplying the value for B in 1, 2, and 3-ft flume widths, simplified expressions of Equation (5) are:

For the 1-ft flume:

$$Q = 6.35 H^{1.321} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7a)$$

for the 2-ft flume:

$$Q = 13.05 H^{1.277} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7b)$$

and, for the 3-ft flume:

$$Q = 19.90 H^{1.245} \quad \ldots\ldots\ldots\ldots\ldots\ldots (7c)$$

The corresponding rating curves are plotted in Fig. 12.

On the basis of computed deviations based on stilling-well readings, it is considered conservative to believe that this type of control flume should be at least as accurate for field measurements, especially of débris-laden flows, as any standard measuring device now in use. However, final conclusions as to its accuracy and the applicability of the general formula must be reserved, pending final rating of the flume in smaller and larger sizes.

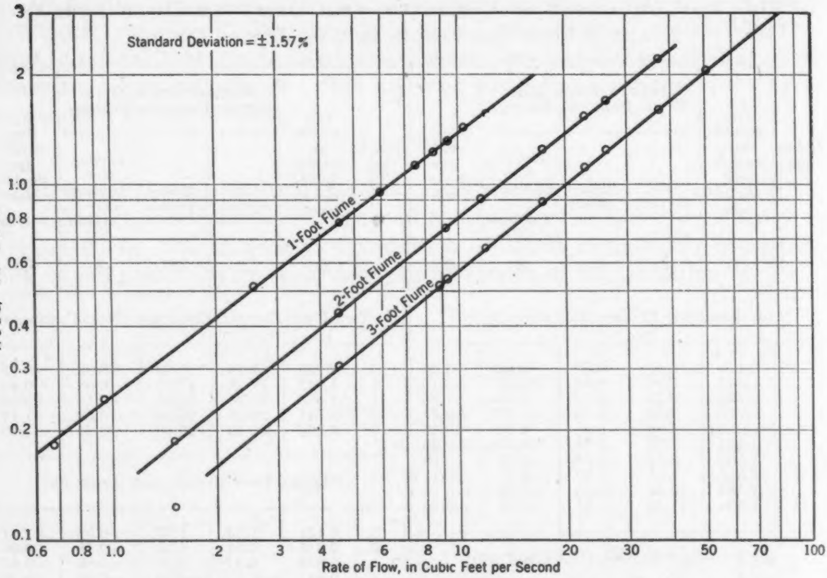

FIG. 12.—RATING CURVES FOR RECTANGULAR, CRITICAL-DEPTH FLUMES

Computed critical depths lie on the curved water surfaces (see Fig. 13) down stream of the transition. A line drawn through these depths is relatively straight, but lies at an angle of approximately 30° from the vertical. These results are similar to the findings of J. G. Woodburn,[9] Assoc. M. Am. Soc. C. E., when he tested broad-crested weirs with a crest slope of 2.6 per cent. In the San

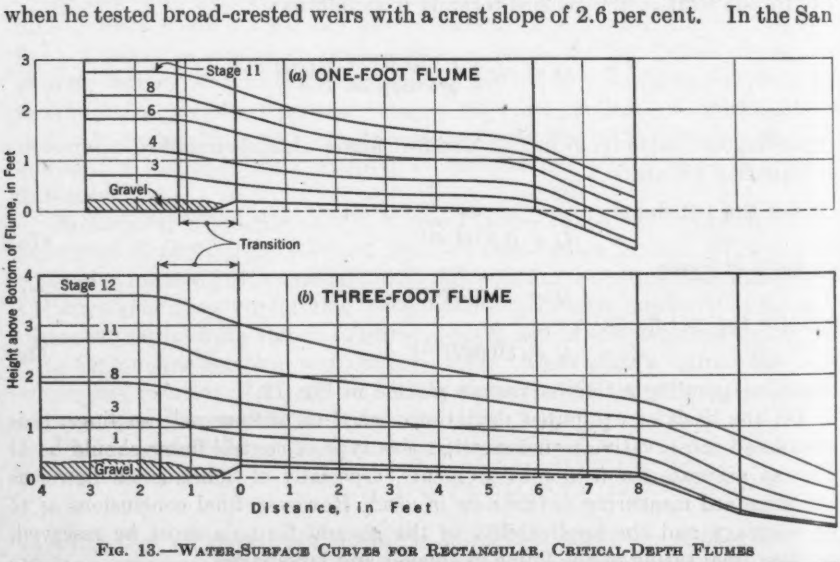

FIG. 13.—WATER-SURFACE CURVES FOR RECTANGULAR, CRITICAL-DEPTH FLUMES

[9] "Tests of Broad-Crested Weirs," by J. G. Woodburn, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 403.

Dimas flume, true critical depths, as indicated by the inversion of water-surface curvature (Fig. 13), actually lie closer than computed to a line vertically above the down-stream end of the entrance transition. These locations were roughly verified by computation of the true critical depths, attempting to correct approximately for centrifugal action in curved flow.

The remaining slope from the vertical of a line through true critical depths may be due to shortness of the intake transition. It is believed that critical depths would remain closer to the "crest" if the transition were more gradual. This belief is borne out by the locations of critical depths in the Parshall flume, and on broad-crested weirs with gently sloping ramp approaches, as tested by A. R. Webb, M. Am. Soc. C. E.[10]

The addition of as much débris as could be fed continuously to these flumes had very slight effect on their ratings (see Table 1). All grades of material from sand to angular stones as large as 10 in. in diameter passed through rapidly, with no deposition on the floor. A 4-ft San Dimas flume has been in operation during the rainy seasons of 1935–36 and 1936–37 in a stream in the Experimental Forest which carries bed-loads estimated at quantities up to 10% of the flow. This flume has remained clean of débris deposits and has shown uniform flow characteristics throughout this period. A 2-ft flume, similarly installed, has given excellent records for one season, in a stream carrying heavy loads of material eroded from road fills.

Flow in the experimental flumes was not affected by velocity of approach or by back-water causing a jump within the flume, as long as the jump remained down stream of the stilling-well inlet. In one test, the stream bed up stream of the flume was built up with gravel to a 25% grade. Although there were shooting velocities in this channel section, a jump occurred above the transition, a hydraulic drop within the cylindrical entrance, and measured heads in the flume were identical with those recorded under standard conditions. A hydraulic jump caused by obstacles held in the flume exit also had no effect on depths at the stilling-well inlet.

Trapezoidal Cross-Section, 3% Flume Floor.—These flumes, tested in bottom widths of 0.5, 1.0, and 2.0 ft, exhibited characteristics similar to those of the rectangular flume, in that flow within the flumes remained well below critical depths at all stages measured, and the addition of bed-load apparently had little or no effect upon their rating. However, unstable flow conditions, expressed in surface waves, persisted in this shape of flume at all stages and in all three flume widths. The result was undesirable variation of observed flow from computed flow values. In Table 3, the equations for the 6-in., 1-ft, and 2-ft flumes were:

For the 6-in. flume,

$$Q = 4.40H^{1.53} \qquad (8a)$$

for the 1-ft flume,

$$Q = 9.60H^{1.45} \qquad (8b)$$

and, for the 2-ft flume,

$$Q = 15.10H^{1.45} \qquad (8c)$$

[10] "Supplemental Tests—Weirs with Aprons Inclined Up Stream and Down Stream," by A. R. Webb, *Transactions*, Am. Soc. C. E., Vol. 96 (1932), pp. 408–416.

These fluctuations may have been caused by over-abruptness of the conical entrance transition.

Critical depths deviated in this type of flume just as far from the vertical as in the rectangular flume.

TABLE 3.—COMPARISON OF COMPUTED WITH OBSERVED DISCHARGES FOR 6-INCH, 1-FOOT, AND 2-FOOT CRITICAL-DEPTH FLUMES: TRAPEZOIDAL CROSS-SECTION

Reading No.	Observed upstream head, H_a, in feet	Rate of Flow, Q, in Cubic Feet per Second			Percentage deviation
		Observed	Computed	Deviation (Column (2) − Column (3))	
	(1)	(2)	(3)	(4)	(5)
(a) SIX-INCH FLUME: (EQUATION (10a))					
3	0.326	0.802	0.792	−0.010	−1.2
2	0.481	1.389	1.434	+0.045	+3.2
1	0.932	3.918	3.951	+0.033	+0.8
4	1.509	8.488	8.272	−0.216	−2.5
(b) ONE-FOOT FLUME: (EQUATION (10b))					
3	0.172	0.802	0.749	−0.05	−6.6
2	0.266	1.389	1.411	+0.02	+1.4
1	0.559	3.918	4.137	+0.22	+5.6
4	0.947	8.488	8.880	+0.39	+4.6
5	1.041	9.86	10.18	+0.32	+3.2
6	1.112	10.98	11.19	+0.21	+1.9
7	1.269	13.73	13.56	−0.17	−1.2
8	1.464	17.24	16.68	−0.56	−3.2
(b) ONE-FOOT FLUME: (EQUATION (10b)) (*Continued*)					
9	1.777	22.97	22.10	−0.87	−3.8
10	2.177	31.26	29.60	−1.66	−5.3
(c) TWO-FOOT FLUME: (EQUATION (10c))					
3	0.125	0.802	0.740	−0.06	−7.5
2	0.192	1.389	1.374	−0.02	−1.4
1	0.390	3.918	3.850	−0.07	−1.8
12	0.480	5.290	5.210	−0.08	−1.5
4	0.682	8.488	8.668	+0.18	+2.1
5	0.762	9.86	10.18	+0.32	+3.2
6	0.816	10.98	11.25	+0.27	+2.5
7	0.944	13.73	13.89	+0.16	+1.2
8	1.094	17.24	17.20	−0.04	−0.2
9	1.324	22.97	22.70	−0.27	−1.2
10	1.631	31.26	30.70	−0.56	−1.8
11	2.490	58.09	56.70	−1.39	−2.4

SUMMARY OF CONCLUSIONS

Conclusions drawn from these experiments may be summarized briefly as follows:

(1) Construction modifications used in Parshall flumes on the San Dimas area have a negligible effect on their rating. The results indicate that the Parshall flume gives accurate measurements of clear water or flows containing loads of fine material, and is reasonably accurate for measuring large flows with bed-load.

(2) Presence of bed-load substantially influences the rating of Parshall flumes at low stages, although the resulting errors at high flows appear to be small.

(3) No alterations introduced into the Parshall flume succeeded in materially improving its accuracy in measuring small and intermediate flows containing bed-load.

(4) A new design of critical-depth flume was developed, which appears to give accurate measurements of discharge and to be unaffected by velocity of

approach or presence of bed-load. This flume functions as a broad-crested weir in which a cylinder-quadrant transition is used in place of the conventional contraction. Depth measurements are made down stream of the "critical" section, where rapid flow occurs. A tentative rating has been established for three widths and a general formula computed.

(5) Use of this type of flume in conjunction with the present San Dimas stations should make possible the measurement, with reasonable accuracy, of small and intermediate loaded flows which at present fail to be measured because of débris deposition in weir boxes and in the Parshall flumes.

Further Experimentation

Completion of experiments with the San Dimas flume involves model studies to perfect its design, and more detailed quantitative investigation of the flow characteristics of the flume in transportation of bed-loads. The model studies are largely completed, and the results will appear in the closing discussion. For bed-load studies an experimental flume structure is being constructed, in which flows containing up to 25% of bed-load material can be circulated continuously.

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PAPERS

PRE-STRESSED REINFORCED CONCRETE AND ITS POSSIBILITIES FOR BRIDGE CONSTRUCTION

By Ivan A. Rosov,[1] M. Am. Soc. C. E.

Synopsis

This paper contains a brief citation of uses made of pre-stressed steel reinforcement in the past and the development of formulas for the solution of special cases. The symbols are defined where they first occur and are summarized for reference in the Appendix.

Introduction

It is customary to consider that the complex, non-homogeneous nature of reinforced concrete is its greatest disadvantage. It is impossible to utilize the strength of steel and of concrete, in the same structural member, with the same degree of economy. In parts of a structure where compression predominates the steel is always understressed, and when tension predominates the designer nearly always assumes that the steel resists all of it. The tensile stress in concrete often exceeds the ultimate, and the resulting cracks are a particularly undesirable characteristic of reinforced concrete. Although such cracks may not immediately endanger the stability of the structure (since the steel resists all the tension), they permit the structure to be destroyed gradually—the concrete by chemical reaction and the steel by corrosion.

On the other hand, the very difference between the two materials can be utilized to the advantage of reinforced concrete structures over all-steel structures. The reinforcement can be designed so as to confine the deformation of concrete within reasonable limits, or, conversely, to produce stresses in the concrete that will act in the proper direction. If it is desired to counterbalance some of the deformations in steel that are expected to occur under the action of working loads, a preliminary stress may be imposed upon the concrete.

This idea of restraining the effects of stress in either material has been accepted as a possibility since reinforced concrete was first introduced, and several structures have been built according to this principle. Only in later years, however, have designers begun an extensive and far-reaching study of all

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the possibilities for the purpose of utilizing it consistently in structural design and construction. One method of restraining deformations may be illustrated by the case of a concrete column with spiral reinforcement, which, for all practical purposes, is the same as a steel "envelope" that prevents the member from bulging laterally under axial loads. The same effect can be produced by steel reinforcement arranged transversely in a structural member so that its bond with the concrete will serve to establish resistance to deformation in that direction. In this case the external restraining forces exerted by the "envelopes" are replaced by the internal forces in the steel. The workability of this principle has been proved by the results of tests[2] conducted by M. Caquot and P. Brice. These investigators found that any type of transverse reinforcement increases the ultimate compressive strength of concrete in so far as it tends to retard lateral bulging due to axial pressure. Similarly, E. Freyssinet[3] has made concrete that resisted, safely, the lateral bulging caused by axial pressures of 3 550 lb per sq in. He found also that the best type of reinforcement for this purpose is two series of parallel wires set in planes normal to the axis of the loads and tied by semi-circular steel bars.

Another general method of changing deformations of either material in a desired manner may be described broadly under the term, "pre-stressing." A brilliant demonstration of the advantages of this idea is afforded by experience gained in constructing the Albert Louppe Bridge, in France,[4] and the bridge across the Rogue River, in the United States.[5] Pressure exerted by means of jacks at key points along the arches had the effect of eliminating the shrinkage stresses and this decreased, materially, the dimensions of the arches.

The principle of pre-stressing was used by A. Mesnager and T. Vevriers in 1930,[6] and, in this case, was applied both to the concrete and to the steel separately. The arches of the bridge[6] at Vesinet, France, are reinforced by steel tubes, thick enough to constitute a considerable percentage of the cross-sectional area. Ordinarily, shrinkage stresses would have stressed the steel to 12 000 lb per sq in., but, by exerting a pull on the tubes and, simultaneously, a pressure on the concrete within the tubes, the shrinkage stresses were canceled. In other words, when the concrete had shrunk, the preliminary stresses in the steel tubes had disappeared, and the full strength of the metal was utilized to resist loading.

The next progressive step would be to increase the preliminary stresses sufficiently to have a residual stress even after the shrinkage stresses have canceled it partly. This would have the effect of introducing known internal stresses in a structure at previously (and precisely) determined points.

An example of the manner in which this principle is applied to reservoirs was published[7] in 1933. The circular concrete wall of the reservoir is fitted with a series of circumferential rings, and these rings finally are covered with a layer of concrete, leaving the connecting turnbuckles revealed. As the water rises in

[2] *Structural Engineer*, June, 1931.
[3] *Memoires* de la Société des Ingénieurs Civils de France, July-August, 1930.
[4] *Le Génie Civil*, October 4, 1930.
[5] *Engineering News-Record*, 1931, No. 26.
[6] *Le Génie Civil*, January 17, 1931; also, *Engineering and Contracting*, August, 1931.
[7] *Engineering News-Record*, February 16, 1933.

the reservoir tensile stresses develop in the concrete, which are counterbalanced by the pressure introduced in the steel hoops by means of the turnbuckle. In this manner the compression may be made to balance the tension in the concrete, thus safeguarding the structure against the effects of cracking and leaking.

This type of structure, resembling ordinary wooden tanks in principle, has recently been used quite extensively,[8] and with good results. The bond between the concrete and the steel is destroyed by coating the latter with a special asphaltic compound and the circumferential rods slip freely while strained by turnbuckles.[9] The same idea has been used for pipes, with equal success.

In Europe, the practice of prestressing does not involve destroying the bond, which conserves the character of the reinforced concrete as a structural unit and distributes the effect of the prestressing uniformly all over the length of the structure. In this manner, Professor F. Emperger applied the principle of the prestressing to precast pipes, and E. Freyssinet to pipes, posts, foundations etc.

It should be possible to prevent deformation in many other kinds of structures, by pre-stressing the materials of which they are composed. E. Freyssinet has stated his conclusion that reinforced concrete bridges of spans as great as 3 300 ft are possible in this manner. The development of structures of this kind may not be the problem of most immediate concern, but in ordinary engineering practice there is a wide field for economy and improvement in design by a proper application of the pre-stressing principle.

Pre-Stressed Bridge Structures

Only simple slabs, beams, and girders are treated in this paper, because they are the structural elements that lend themselves most easily to pre-stressing, and in which the results are obvious and most interesting. The reinforcement along the tension face is straight and can be stretched by means of a simple device before the concrete is poured. After the concrete has set the tension is removed from the reinforcement, which then tends to return to its normal length. The result is that, through the bond between the reinforcement and the concrete, an internal force is created which, in turn, resists the deformation due to the application of external forces when the structural member is put to its final use. With respect to the concrete alone, of course, the bond in the steel constitutes an external force, and the amount of this force that remains after the concrete has set may be computed if the relation, $n = \frac{E_s}{E_c}$, is known.

The effect of pre-stressing is opposite to that of the final loading (see Fig. 1). Obviously, it is possible to pre-stress a structural member in such a way that the working load would never produce stresses great enough to cancel the internal stresses entirely. In that case, the cross-section of the beam (see Fig. 1(*c*)) would be entirely in compression.

Since there is no tension in the beam, any cracks that tend to form as a result of shrinkage, or other causes, are automatically closed by the pressure of

[8] *Engineering News-Record*, April 16, 1936.

[9] "The Preload System of Concrete Construction," by J. M. Crom.

the steel reinforcement. The net effect is to eliminate the most serious causes of deterioration in concrete, namely, penetration of water or gas and the rusting of the steel reinforcement. Furthermore, the entire cross-section of the concrete becomes effective in compression. Under such ideal conditions the

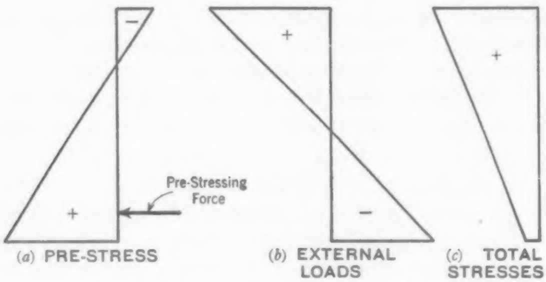


FIG. 1.—STRESSES IN A PRE-STRESSED BEAM

dimensions of reinforced concrete members may sometimes be decreased as much as 50 per cent. The dead load is much more important in concrete structures than in steel frame structures. For larger spans the weight of a bridge, for example, increases so rapidly as to prohibit the use of reinforced concrete. By pre-stressing the reinforcement, as suggested herein, thus reducing the sectional dimensions appreciably, the limiting span for reinforced concrete structures can be increased considerably.

The foregoing ideas, as applied to the introduction of initial stresses in a beam, are not new. As early as in 1907 Lund and Koenen[10] proposed placing the tensile steel in tension before placing the concrete. Bending tests with beams submitted to an initial tension of 8 500 lb per sq in. showed that the first cracks appeared when the loads exceeded by about 50% those corresponding to beams made in the ordinary manner; but the efficacy of the beams decreased greatly as time passed.

Some other attempts also failed because the preliminary stresses in the steel were not sufficient to create the desired results. Steel that has a yield point from 33 000 to 40 000 lb per sq in., ordinarily has an assumed allowable stress from 16 000 to 20 000 lb per sq in., so that the bars cannot be stressed to more than 10 000 lb per sq in. In some cases this degree of pre-stressing could not even offset the effect of accidental errors. Even the largest permissible stress imposed on the reinforcement, in such cases, would require too much steel in order to create internal stresses in the concrete sufficient to afford a saving in material. In later years, however, steel has been developed with an ultimate strength of as much as 140 000 lb per sq in., with yield point stresses as great as 110 000 lb per sq in., and the prices for this high-strength steel differ only a trifle from those of ordinary structural steel. By hardening and annealing a cold-drawn steel, and then cold drawing it again, it is possible to raise the yield point to from 185 000 to 198 000 lb per sq in.

In steel structures the alloys that are brittle and lack plasticity are avoided. Steel members with large and irregular sections can be damaged during the

[10] *Le Génie Civil*, No. 9, 1936.

cooling if the alloys are too brittle. In concrete structures it is the concrete that provides the plastic element to the structure. On the other hand, small cross-sectional dimensions of reinforcement bars prevent damage, when such bars are cooled.

E. Freyssinet prestresses reinforcement so as to impose tension to between one-half and one-third of the elastic limit of the metal. A rod with a yield point of 120 000 lb per sq in. may be prestressed safely to as much as 80 000 lb per sq in., and with material of this high strength it is possible to utilize the advantages of prestressing for bridge construction. However, as far as the writer is aware, the only actual attempt in this direction has been the construction of a large-scale model of a girder by a German concern which, in 1936, tested a model 65.5 ft long.

Basic Requirement of the Pre-Stressed Design

The most important difference between an ordinary design of reinforced concrete and the "pre-stressed" design is that in the latter all the concrete in any cross-section may be kept in compression.

If external forces produce tensile stresses, they are treated merely as negative stresses, serving to decrease the compression created by the pre-stressed reinforcement. The entire cross-section of the concrete is considered effective in compression. The imposition of initial stresses should be considered in three successive stages, as follows:

Stage (1).—Before concrete is poured an external tensile force is applied mechanically to the reinforcement. This force may be termed the "preliminary" tension. After concrete has been poured, the effect of shrinkage is to restore the length of the deformed bar to a certain extent, thus releasing a part of the preliminary tension. On the other hand, the concrete is not permitted to shrink to its fullest extent, and bond stresses are created between the steel and the concrete, which have the effect of introducing an eccentric external force. Stresses from this source are not great, compared with those due to external loading, and they are not considered in ordinary design. No consideration is needed, and for the pre-stressed beam, the period in which the concrete is setting is the only time in which it is under slight tension.

Stage (2).—After the concrete has set, the preliminary tensile force is removed from the reinforcement, which then tends to shorten to its original length and is prevented from doing so only by the concrete; in this process, the shrinkage stresses are eliminated, leaving a residual eccentric force to act against the external forces that may be applied to the beam later. This eccentric force produces a direct pressure and a bending moment in the beam. If the bottom reinforcement is prestressed the effect is to introduce tension at the top, which, in turn, is offset by the dead weight of the beam; but before the initial pull is released from the reinforcement, this dead load is supported by forms and timbering. Upon release the beam bends upward, becoming loosened from the forms, and rests on its ends. In this stage the stresses in the beam are a combination of pre-stresses, f_p, and dead load stresses, f_D.

In the design of a prestressed reinforced concrete beam, the aim is to eliminate net tension in the entire cross-section and to confine the compression

within allowable limits. Expressed algebraically, the following conditions must be satisfied at Stage (2):

(*A*) At the top of the beam:

$$f = f_{pt} + f_{Dt} > 0$$

(*B*) At the bottom of the beam:

$$f = f_{pb} + f_{Db} < f_{ac}$$

in which f = the combined or net unit stress; f_t = the unit stress at the top; f_b = the unit stress at the bottom; and f_{ac} = the allowable unit stress in concrete. In other words, the initial tension at the top of the beam should be absorbed by the compression due to dead load, and the initial compression exerted by the pre-stressed steel on the concrete at the bottom of the beam should be reduced to within the allowable value, f_{ac}, for concrete, by the dead load.

Stage (3).—The final stage is that in which the beam carries the full live load; the beam having been prestressed to offset the effects of dead load, it must have an excess of prestress to cancel the stresses produced by live loads. For this purpose the following conditions must be satisfied:

(*C*) At the top of the beam:

$$f = f_{pt} + f_{Dt} + f_{Lt} < f_{ac}$$

(*D*) At the bottom of the beam:

$$f = f_{pb} + f_{Db} + f_{Lb} > 0$$

(*E*) In the steel at the bottom of the beam:

$$f_s = f_{ps} + f_{Ds} + f_{Ls} < f_{as}$$

in which f_L = the unit stress due to live load; and, f_s = the unit stress in the steel.

To take full advantage of the methods described herein, the beam should be loaded temporarily up to the full dead loads before the preliminary stress in the reinforcement is released. Otherwise, the prestress in the reinforcement would be kept too low, limited by Conditions (*A*) and (*B*).

In analyzing the stresses in the reinforcement, two distinct factors must be taken into account: (*a*) The preliminary stress; and (*b*) the "designed prestress," f_{ps}, remaining after the partial loss of the "preliminary stress." This loss is due to the shrinkage, to the plastic flow, and to the compressive deformations of the surrounding concrete.

It is important to determine the "designed prestress" exactly. The loss of the preliminary stress, due to the shrinkage, depends on the quality of the concrete. For ordinary concrete the shortening may exceed 0.0004. This is equivalent to a decrease of 12 000 lb per sq in. in the tension of the steel. The shrinkage of compact concrete is much less.

The plastic flow should also be considered because the structure will be subject to permanent pressure. After 1 yr the plastic flow[11] for ordinary

conditions can reach a value equal to $0.13 \times \sqrt{365} \times 0.000001$, or 0.0000009 per lb per sq in. of stress. The final flow after 5 yr will exceed this amount by not more than 30 to 40 per cent. Thus, if the permanent pressure in the concrete is 500 lb per sq in., the shortening of the concrete under plastic flow may reach $0.0000009 \times 1.4 \times 500 = 0.0006$, with a corresponding decrease of 18 000 lb per sq in. in the prestress.

A change in the prestress will be produced by the prestress itself while it is in the process of compressing the concrete. The reinforcement follows the shortening of the compressed concrete and a fraction of its stress is released. If the compression of the concrete around the bars is 500 lb per sq in., the decrease in the tension of the steel will be about 5 000 to 7 500 lb per sq in.

Thus, in time, the "preliminary stress" may lose as much as 12 000 + 18 000 + 5 000 = 35 000 lb per sq in. and only the remaining tension in the steel should be taken into account for the design. This remainder is the "designed prestress." However, the structure should also be checked to meet the conditions of the first days, with a partial shrinkage and no plastic flow, to be sure that this temporary excess of the prestress will not produce an undesirable effect.

The stresses due to dead and live load will be governed by a factor of safety of 2. Ordinarily, these stresses are small compared to the designed prestress and range from 8 000 to 12 000 lb per sq in. Even with a factor of safety they cannot usually restore the loss in the preliminary stress.

Thus, the "designed prestress" may be fixed safely by deducting from 30 000 to 40 000 lb per sq in. from the yield-point value of any given steel. The "preliminary stress" should be from 5 000 to 10 000 lb per sq in. less than the yield point. The excess required for the "preliminary stress" above the "designed prestress" should be estimated as exactly as possible for a given structure.

As to the concrete, an ultimate strength of 2 000 to 2 500 lb per sq in. in compression is assumed in the examples of this paper. However, if the advantages of pre-stressing are to be fully utilized, special high-strength grades of concrete should be manufactured. Concrete of the highest strength can be produced most readily at a fixed central manufacturing plant, and, consequently, pre-cast beams and slabs can utilize the pre-stressing principle to the greatest advantage. Properties desirable in concrete are that: (1) It deforms at a fairly constant rate; (2) its total shrinkage is relatively slight; and (3) it possesses high strength and quick-hardening properties. The best method of obtaining these qualities is to manufacture a concrete with a minimum quantity of water and to insure that the product is placed carefully, vibrating the forms during the process. E. Freyssinet utilizes specially "treated" concrete, that is, concrete that is vibrated, pressed, and heated during setting. This improves the properties of the concrete to a remarkable extent; in tests conducted by M. Freyssinet the ultimate strength of the concrete made under pressure attained a breaking stress of 14 000 lb per sq in., and more, and a tensile stress of 150 lb per sq in. The "treated" concrete is compact, subject to only slight shrinkage, resists repeated stresses, and is suitable for prestressing.

[11] *Journal*, Am. Concrete Inst., November, 1936.

In all ordinary structures no factor of safety should be assumed; if the actual working stresses exceed the computed stresses, the resultant tension will be relatively slight and will be eliminated subsequently as the structure is restored to a normal state of equilibrium. In special cases, where it is necessary that the design be particularly conservative, allowable working stresses due to live load may be increased by some factor of safety.

Theory of Pre-Stressed Rectangular Beams

In a simple pre-stressed reinforced concrete beam there are bending moments, M, shearing forces, V, and the horizontal force, T, introduced through the reinforcement, to be considered. All the concrete in any cross-section is considered as acting in compression. The cross-section of steel may be replaced, for purposes of computation, by the equivalent concrete area as in ordinary reinforced concrete design.

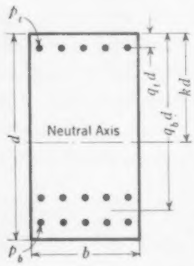


Fig. 2

Referring to Fig. 2, let p = the percentage of steel in the entire cross-section of a beam (p_t referring to the steel at the top of the beam and p_b to the steel at the bottom); d = depth of a beam; and b = width of a beam. Furthermore, let the distance from the top of the beam to the centroid of the top reinforcement be $q_t\, d$, and to the centroid of the bottom reinforcement, $q_b\, d$. The distance down to the neutral axis of the section is $k\, d$, and the effective area of the beam is:

$$A = b\, d + (n - 1)(p_t + p_b)\, b\, d = b\, d\,[1 + (n - 1)(p_t + p_b)] = K\, b\, d \quad \ldots (1)$$

Taking moments about the top of the beam:

$$A\, k\, d = \frac{b\, d^2}{2} + (n - 1)(p_t\, q_t\, d + p_b\, q_b\, d)\, b\, d \quad \ldots\ldots\ldots\ldots (2)$$

from which,

$$k = \frac{\frac{1}{2} + (n - 1)(p_t\, q_t + p_b\, q_b)}{K} \quad \ldots\ldots\ldots\ldots (3)$$

The moment of inertia of the section about the neutral axis, after simplifying, is:

$$I + bd^3 \left\{ \frac{1}{12} + \left(\frac{1}{2} - k\right)^2 + (n - 1)[p_t(k - q_t)^2 + p_b(q_b - k)^2] \right\} = C\, b\, d^3 \quad \ldots\ldots\ldots (4)$$

Finally, the stresses due to the working loads are:

$$f_t = \frac{M}{I}\, k\, d = \frac{k\, M}{C\, b\, d^2} \quad \ldots\ldots\ldots\ldots (5)$$

$$f_b = \frac{M}{I}\,(d - k\, d) = -\frac{(1 - k)\, M}{C\, b\, d^2} \quad \ldots\ldots\ldots\ldots (6)$$

and,

$$f_s = -\frac{M\, n}{I}\,(q_b\, d - k\, d) = -\frac{M\, n(q_b - k)}{C\, b\, d^2} \quad \ldots\ldots\ldots\ldots (7)$$

If there is no top reinforcement, the expressions for k and C become correspondingly simplified. In the case of a slab of unit width, $b = 1$ in Equations (5), (6), and (7).

The pre-stresses are determined in a similar manner. Thus, subtracting the area of the bottom reinforcement, the effective area resisting the pre-stresses is:

$$A_1 = b\,d + (n - 1)\,p_t\,b\,d - p_b\,b\,d = b\,d\,K_1 \qquad (8)$$

Taking moments about the top of the beam, and simplifying:

$$k_1 = \frac{1}{K_1}\left[\frac{1}{2} + (n - 1)\,p_t\,q_t - p_b\,q_b\right] \qquad (9)$$

The moment of inertia of the section about the neutral axis, after simplifying, is:

$$I_1 = b\,d^3\left[\frac{1}{12} + \left(\frac{1}{2} - k_1\right)^2 + (n - 1)\,p_t\,(k_1 - q_t)^2 - p_b(q_b - k_1)^2\right] = C_1\,b\,d^3 \qquad (10)$$

Stresses produced by the eccentric force, P, consist of uniformly distributed compression, $\frac{P}{A_1}$, and the bending stress, which varies from $-\frac{P\,d(q_b - k_1)}{I_1}\,k_1\,d$, at the top to $+\frac{P\,d(q_b - k_1)}{I_1}\,(d - k_1\,d)$, at the bottom. The area of the bottom reinforcement is $p_b\,b\,d$, and the value of the pre-stressing force is,

$$P = -f_{ps}\,p_b\,b\,d \qquad (11)$$

Consequently, the pre-stress at the top of the beam is expressed by:

$$f_{pt} = P\left(\frac{1}{A_1} - \frac{q_b - k_1}{I_1}\,d^2\,k_1\right) = -f_{ps}\,p_b\left(\frac{1}{K_1} - \frac{q_b - k_1}{C_1}\,k_1\right) \qquad (12)$$

and the pre-stress at the bottom is:

$$f_{pb} = P\left[\frac{1}{A_1} + \frac{(q_b - k_1)(1 - k_1)\,d^2}{I_1}\right] = -f_{ps}\,p_b\left[\frac{1}{K_1} + \frac{1}{C_1}(q_b - k_1)(1 - k_1)\right] \qquad (13)$$

If there is no top reinforcement, the factors, C_1, K_1, and k_1 in Equations (9), (10), (12), and (13) become:

$$C_1' = 0.083 + (0.5 - k_1)^2 - P_b(q_b - k_1)^2 \qquad (14)$$

$$k_1' = \frac{1}{K_1}\left(\frac{1}{2} - p_b\,q_b\right) \qquad (15)$$

and

$$K_1' = 1 - p_b \qquad (16)$$

As before, b = unity in the case of a slab. As an approximation, the effect

of subtracting the area of the bottom steel may be neglected, with the following results:

$$C_1'' = 0.083 + (0.5 - k_1)^2 + (n - 1)\, p_t(k_1 - q_t)^2 \quad \text{......(17)}$$

$$k_1'' = \frac{1}{K_1''}\left[\frac{1}{2} + (n - 1)\, p_t q_t\right] \quad \text{......(18)}$$

and

$$K_1'' = 1 + (n - 1)\, p_t \quad \text{......(19)}$$

If the member has no top reinforcement, $p_t = 0$, and Equations (17), (18), and (19), are simplified accordingly. Substituting these simplified values in Equations (12) and (13):

$$f_{pt} = 2\, f_{ps}\, p\, (3\, q - 2) \quad \text{......(20)}$$

and

$$f_{pb} = -\, 2 f_{ps}\, p\, (3\, q - 1) \quad \text{......(21)}$$

As the design of pre-stressed beams and slabs is confined mostly to cases in which there is reinforcement on one side of the beam only, and as the effect of ignoring the steel area is slight, Equations (20) and (21) will be the formulas most commonly used.

The bending capacity of the prestressed beam can be determined from Conditions (C) and (D). For the most part, the first condition governs, that is,

$$\frac{k}{C}\,\frac{M}{b\, d^2} - 2 f_{ps}\, p\, (3q - 2) \gtreqless f_{ac}$$

Hence,

$$M \gtreqless [f_{ac} + 2\, f_{ps}\, p\, (3q - 2)]\,\frac{C}{k}\, b\, d^2$$

The bending capacity of the conventional reinforced concrete beam is,

$$M' = f_{ac}\,\frac{k'\, j\, b\, (qd)^2}{2}$$

and the ratio,

$$\frac{M}{M'} \gtreqless \left[1 + 2\,\frac{f_{ps}}{f_{ac}}\, p\, (3q - 2)\right]\frac{C}{k}\,\frac{2}{k'\, j\, q^2} \quad \text{......(22)}$$

Assuming that $f_{ps} = 45\,000$ lb per sq in.; $f_{ac} = 800$ lb per sq in.; $p = 0.009$; $k'\, j = 0.347$; $q = 0.9$; $n = 15$; and $\frac{C}{k} = 0.186$, $\frac{M}{M'} \gtreqless 2.26$. That is, the bending capacity of the prestressed beam is greater about twice that of an ordinary beam.

Bond

Bond stresses between the reinforcement and the concrete develop only when the stress in the steel changes in value from one point to the other, as when the external forces vary. The preliminary stress introduced in the reinforcement is constant from one end to the other. Considering only the bending moment due to the external load, the bond stress between two sections, a distance, Δx, apart is:

$$u\, \Sigma o\, \Delta x = \Delta f_s\, A_s \quad \text{......(23)}$$

in which Σo is the total perimeter of the steel bars, and Δf_s is the change in the steel stress due to the bending moment. As a limiting value,

$$u = \frac{A_s}{\Sigma o} \frac{d f_s}{dx} \qquad (24)$$

In the conventional design, the appearance of cracks must be anticipated, in which case all the tension is carried by the steel, to balance the compression in the concrete at the top of the section. In that case, $f_s = \frac{M}{A_s \, j \, d_1}$, and,

$$u_1 = \frac{1}{\Sigma o \, j \, d} \frac{dM}{dx} = \frac{V}{j \, d_1 \, \Sigma o} \qquad (25)$$

which is the well-known expression for bond. When the steel has been properly prestressed, however, the concrete over the entire section is always under compression.

The tension induced by the external loads, through the bending moment, is distributed over the concrete area as well as over the steel, with the result that the steel stress, and, consequently, the bond stress (which are proportional to Δf_s) are decreased considerably.

Referring to Equation (7) for the value of f_s, the bond is expressed by,

$$u_2 = \frac{A_s \, n \, (q - k)}{\Sigma o \, C \, d^2 \, b} \frac{dM}{dx} = V \frac{A_s \, n \, (q - k)}{\Sigma o \, C \, d^2 \, b} \qquad (26)$$

and the ratio of bond values for the pre-stressed design and the conventional design is:

$$\frac{u_2}{u_1} = \frac{j \, d_1 \, A_s \, n \, (q - k)}{C \, b \, d^2} \qquad (27)$$

Substituting for A_s its equivalent, $p \, d \, b$ and $d_1 = qd$, and simplifying:

$$\frac{u_2}{u_1} = \frac{j \, q \, p \, n \, (q - k)}{C} \qquad (28)$$

which is always less than one. In Example 1 it proves to be 0.53, which indicates that, for this case, the bond stress to be designed for in the pre-stressed beam is only 53% as severe as in the conventional design. In fact, in most cases of pre-stressed beams, the bond to be provided is relatively weak and need not be considered. If it is desired to check the bond stresses the following formula may be used:

$$u = V \frac{p \, n(q - b)}{C \, \Sigma o \, d} \qquad (29)$$

Toward the end of a beam the reinforcement is stressed in a manner similar to that in the vicinity of tension cracks in the conventional design. Theoretically, the bond stress is infinitely great across a crack but, actually, the steel slips, thus gradually transferring excessive stresses to the concrete at that point. As is known, concrete is plastic enough to "flow" under applied load as in the case of columns, the effect being to "repair" changes in the relation between

particles, thus avoiding rupture. This phenomenon has not been studied adequately, but it is extremely important in problems involving the design of reinforced concrete.

In the case of pre-stressed concrete, by which preliminary forces are introduced in the reinforcement, there is a section of the steel at both ends, beyond the point at which the forces were applied, where the induced stresses are equal to zero. Theoretically, the transition between zero pre-stress and full design pre-stress occurs over a very short length of bar, and over this length the bond is infinitely great. As a matter of fact, the distance over which this transition occurs is appreciable because of the fact that the bars slip. E. Freyssinet, who conducted tests with preliminary stresses as high as 256 000 lb per sq in., does not report any difficulties in dealing with this problem. At the joint meeting of the Institution of Structural Engineers and the Société des Ingénieurs Civils de France in 1937, he explained that adhesion is not a question of the concrete sticking to the steel, but that it is a wedging action under the effect of the transversal deformations of the concrete, similar to the gripping of a bar in a wedge clamp. Its intensity and its efficacy depend on the quality of the concrete and the packing of the concrete around the bars. In the case of "treated" concrete, very restricted lengths (approximately 10 or 20 diameters) are sufficient to assure anchorage, provided there is a sufficient thickness of concrete and that the surface of the metal is not smooth in the zone of anchoring.[12] In particular cases, the use of vertical end anchor-plates can be recommended. If the reinforcement is designed to pass through such plates so that the ends are fastened securely at the outside surfaces, the preliminary stress at the end will then be divided between bond and the direct pressure against the concrete at the end section.

Diagonal Stresses

In conventional reinforced concrete design, diagonal stresses are of great importance and a complicated system of special reinforcement is usually designed to offset them. The value of these stresses is expressed as follows:

$$f_d = \frac{1}{2} f_l \pm \sqrt{\frac{1}{4} f_l^2 + v^2} \quad \ldots\ldots\ldots\ldots\ldots\ldots (30)$$

in which f_l and v are the longitudinal stress and the shear, respectively, produced by the bending moment at a given section. If f_l is tension, the diagonal stresses at the point will vary from maximum tension, which will sometimes be considerably greater than the computed longitudinal stress, to a maximum compression which will be comparatively small. The longitudinal stress, f_l, decreases from the center of the beam to the support and from the bottom of the beam to the neutral axis, but combined with the effect of the shear, v, this stress can produce diagonal tension great enough to be unsafe. Consequently, diagonal reinforcement is designed to meet this condition. In the pre-stressed beam, conditions are entirely different. The longitudinal stress is always compressive, and the second term of Equation (30) gives usually only a negligible excess above the first term.

For an approximate estimation, the second term may be replaced by the two first terms of the corresponding series because, for the prevailing values of

[12] *Structural Engineer*, May, 1937.

f_l and v, the series will be converging. Then,

$$f_d = \frac{1}{2} f_l - \left(\frac{1}{2} f_l + \frac{v}{f_l} \right) = - \frac{v}{f_l} \quad \text{.............. (31)}$$

In the prestressed design f_l varies from f_{ac} at the top to zero or some positive value at the bottom, making at the neutral axis about $\frac{1}{2} f_{ac}$. For $f_l = 300$ lb per sq in. and $v = 150$ lb per sq in., the diagonal tension will be only $- \frac{150}{300} = - \frac{1}{2}$ lb per sq in., which is absurd.

Thus, the diagonal stresses need not be considered. They eliminate the bending of the longitudinal bars and considerably simplify the reinforcement. The use of stirrups, however, is desirable, because the prestressed beam is under permanent compression and the stirrups will play the rôle of hoops in columns. Correspondingly, stirrups should form rings restraining the effect of the bulging in concrete and should be placed along the entire span of the beam.

If a particular structure requires absolutely no tension in the concrete, the stirrups near the support may by prestressed as well.

Deflection

The prestressing design introduces a direct pressure and new bending moment in addition to the bending moment of loading. The direct pressure uniformly distributed over a transverse section does not affect the deflection. The "prestress" bending moment acts in the direction opposite the loading and induces an upward deflection. Therefore, it may be stated that the prestress decreases the deflection of a beam.

In the customary balanced design, with the unit stress at the top of the concrete, f_{ac}, and at the reinforcement, f_{as}, the unit length of the top fiber will decrease by $a = \frac{f_{ac}}{E_c}$ and the unit length of the reinforcement will increase by $b = \frac{f_{as}}{E_s}$. This tends to bend the beam and the radius of curvature, r, referred to the top of the beam, will be computed from the equation,

$$\frac{r + q\,d}{r} = \frac{1 + b}{1 - a} \quad \text{.................... (31a)}$$

in which $q\,d$ is the distance between the top fiber and the reinforcement.

Transformation of Equation (31a) gives,

$$r = q\,d\,\frac{1 - a}{a + b} \quad \text{..................... (31b)}$$

In the "prestress" design, the unit length of the top fiber will decrease by a, as a maximum, and the bottom fiber will decrease too, or, in the extreme case, will not change. As before,

$$\frac{r_1 + d}{r_1} = \frac{1}{1 - a} \quad \text{..................... (31c)}$$

Hence,

$$r_1 = d\,\frac{1-a}{a} \qquad (31d)$$

The ratio of both radii is,

$$\frac{r_1}{r} = d\,\frac{1-a}{a} \times \frac{a+b}{q\,d(1-a)} = \frac{a+b}{qa}$$

Assuming that f_{ac} = 800 lb per sq in.; f_{as} = 20 000 lb per sq in.; E_c = 2 000 000 lb per sq in.; and, $E_s = 15\,E_c$; $q = 0.9$; $a = 0.0004$; $b = 0.00067$; and $\frac{r_1}{r} = 2.95$. On the other hand, the prestressed beam can carry about twice its normal load. If equal loads are considered, the deflection for the prestressed beam will be from five to six times less than for the conventional beam.

Peculiar Form of Prestressed Beams

To eliminate the tensile stresses completely, Conditions (*A*) to (*D*) must be satisfied at every point along the span. If the section of the beam is designed in such a manner that these conditions are satisfied for the center of the span, it may be unsatisfactory for other points of the span where the outside bending moment is less than the maximum. Indeed at the top of the beam, the compression due to dead load (which must absorb the tension created by the prestressing) decreases to zero at the support where the outside bending moment is zero. Under such conditions, there could be a residual preliminary tension at the top of the beam near the support where Condition (*A*) would not be satisfied. Similarly, residual preliminary compression along the bottom of a beam may not have been adjusted to the allowable limit for compression with the result that Condition (*B*) would not be satisfied.

This difficulty can be overcome easily by varying the section. The bending moment due to dead load varies along the span according to the parabolic law, $x(1-x)$. To satisfy Condition (*A*), the pre-stress, f_{pt}, should be made to vary according to the same law by changing p or q, or both, in Equation (20). This is accomplished by increasing the width or the height of the beam. The reinforcement under preliminary tension is fixed in position in cross-sectional area and cannot be utilized to adjust the pre-stressed condition.

For simple beams and slabs the best method is to increase the depth, without changing the position of the steel. In this case, the depth, d_x, of a section at a distance, x, from the support, is determined theoretically by the expression (see Equation (20)):

$$2\,f_{ps}\,p\,\frac{d}{d_x}\left(3\,\frac{d}{d_x} - 2\right) = x(1-x)2\,f_{ps}\,p\left(3\,\frac{d}{d_x} - 2\right) \qquad (32)$$

or,

$$\frac{d}{d_x}\left(3\,\frac{d_b}{d_x} - 2\right) = x(1-x)\left(3\,\frac{d_b}{d} - 2\right) \qquad (33)$$

in which $d_b = qd$ is the permanent distance from the top of the beam to the steel. For the ends of the beam, where $x = 0$, or $x = 1$, Equation (33)

reduces to $3\frac{d_q}{h_0} - 1 = 0$; or,

$$d_0 = \tfrac{3}{2}\, d_b \quad \text{.......................}(34)$$

Equation (34) is the algebraic expression for the well-known rule that when an eccentric force is applied at the third point of a section, the stresses at the farthest edge of that section are equal to 0. Condition (*B*) is usually satisfied when the depth is determined by Equation (34). The addition to the depth of a beam decreases rapidly from the supports and for $x = \frac{1}{4} l$, this addition is quite small. It is probably better to select a beam of the type shown in Fig. 3 rather than the parabolic shape dictated by the strict theory.

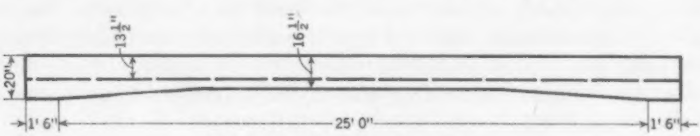


FIG. 3.—PRE-STRESSED SLAB BRIDGE

A second solution consists in lowering the top of the beam so that at the support the reinforcement is at a depth of $\frac{2}{3}\, d$ from the top. The distance from the bottom of the beam to the reinforcement is then represented by $d - q\, d$, and the depth of the beam at the support should be,

$$d' = 3\, d\, (1 - q) \quad \text{.......................}(35)$$

In the case of beams and slabs, Equation (35) yields values for depth which are too small, and difficulties are encountered due to high compressive pre-stresses and shearing stresses near the support. In the case of large girders, however, this solution of the problem is worth considering.

Continuous beams and rigid frames present the same problem to an even greater degree because in this case negative moments occur over the supports. The prestressing must absorb tension at the top and compression at the bottom of the beam. For this purpose the depth of the beam over the support must be increased to $d_b < \frac{2}{3}\, d_0$ or, even if the negative moments are large, to $d_b < \frac{1}{2}\, d_0$. In the latter case the sign of the "prestressed" moment changes and its effect upon the beam reverses.

Consequently, for the continuous beams and rigid frames the middle section is designed as for a simple beam, and the values of d_b and d are computed according to the foregoing equations. At the point of inflection, d increases to the value of $\frac{3}{2}\, d_b$. At the supports, the section is designed for negative moment and the same formulas are simply reversed, changing the top and bottom. In this case, the lowering of the top of the beam toward the support can be of a great use because it augments the relative eccentricity of the prestress.

If the span of a bridge is great and if rectangular beams are not economical, I-shaped girders should be used. In the conventional reinforced concrete design such girders are not used because the bottom flange would be useless inasmuch as it would tend to crack regardless of its dimensions. Consequently, the **T**-beam section has been adopted; but with prestressing the I-shaped girder

becomes as convenient and as economical in reinforced concrete design as it is in structural steel design.

The foregoing theory of the prestressed beam is easily applied to the I-beam shape. Final formulas and conditions for the design are the same and only the coefficients, K, k, and C, differ, according to the special shape of sections.

The method of meeting the change in the outside moments by varying the depth of the beam (that is, the eccentricity of the prestress), is identical with that described herein.

There are cases in which the principle of pre-stressing cannot be applied because it is impossible to select a value for the preliminary stress that will satisfy all the required conditions at the same time. This is true when the live load is considerably greater than the dead load, such as in the case of floor-slabs. The maximum ratio between live load and dead load for purposes of pre-stressing can be determined as follows: By Condition (D) the minimum prestress at the bottom required to absorb the maximum tension due to the loading is expressed by,

$$f_{bp} = -f_{bD} - f_{bL} \quad \text{(36)}$$

Since f_{bL} is equal to $f_{bD}\,\dfrac{L}{D}$, Equation (36) can be written in the form:

$$\frac{f_{bp}}{f_{bD}} = -\left(1 + \frac{L}{D}\right) \quad \text{(37)}$$

On the other hand, by Condition (A), the tensile pre-stress at the top must be absorbed by the compression due to dead load. Consequently, $f_{pt} = -f_{Dt}$; or, $\dfrac{f_{pt}}{f_{tD}} = -1$; from which,

$$\frac{f_{bp}\,f_{tD}}{f_{bD}\,f_{pt}} = 1 + \frac{L}{D} \quad \text{(38)}$$

The relation, $\dfrac{f_{bp}}{f_{tp}}$, between the pre-stress at the bottom and at the top, by Equations (20) and (21), becomes,

$$\frac{f_{bp}}{f_{tp}} = -\frac{3\,q - 1}{3\,q - 2} \quad \text{(39)}$$

and the relation between the stresses at the top and at the bottom, due to dead load, is,

$$\frac{f_{tD}}{f_{bD}} = -\frac{k}{1 - k} \quad \text{(40)}$$

Multiplying Equations (39) and (40):

$$\frac{f_{bp}\,f_{tD}}{f_{tp}\,f_{bD}} = \frac{3\,q - 1}{3\,q - 2}\,\frac{k}{1 - k} \quad \text{(41)}$$

Finally, by combining Equations (38) and (41) and solving for the ratio, $\dfrac{L}{D}$:

$$\frac{L}{D} = \frac{k\,(3\,q - 1)}{(1 - k)\,(3\,q - 2)} - 1 \quad \text{(42)}$$

In other words, if the ratio between live load and dead load exceeds the value represented by the right-hand member of Equation (42), the preliminary stresses in the concrete cannot absorb the maximum tension at the bottom, and, at the same time, the stresses to be absorbed at the top by the minimum compression. Therefore, it is impossible to design the beam for complete elimination of tension in the concrete. At the values of $k = 0.55$, $q = 0.9$, and $\frac{L}{D} \gtreqless 2$.

The following illustrative problem will demonstrate the tremendous saving and improvement in design possible by introducing preliminary stresses.

Example 1.—To demonstrate the manner in which pre-stressing can influence the dimensions of a reinforced concrete structure, consider a pre-cast, slab bridge. At present, the excessive weight of pre-cast slabs limits their use to comparatively small spans, and any method of decreasing the thickness of such slabs would permit increasing the span length; for example, consider a bridge[13] with a clear span of 25 ft composed of separate units 29 ft long and 16 ft wide. Each slab is pre-cast in a central plant, transported to the site, and placed in position by cranes. For a slab 28 in. thick, the effective span may be assumed equal to $25 + 2.33 = 27.33$ ft. The dead load is 415 lb per sq ft, and for an element 1 ft wide the bending moment due to dead load is,

$$M_D = \frac{415 \times 27.33^2}{8} = 38\,600 \text{ ft-lb}$$

The live load consists of a 20-ton truck with one pair of wheels, arranged as shown in Fig. 4, for maximum live load moment. The value of this moment,

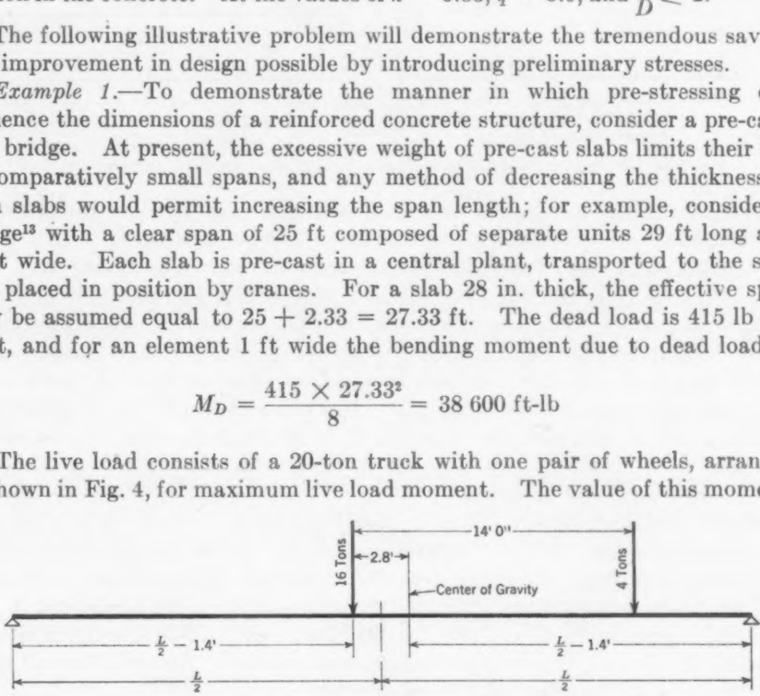


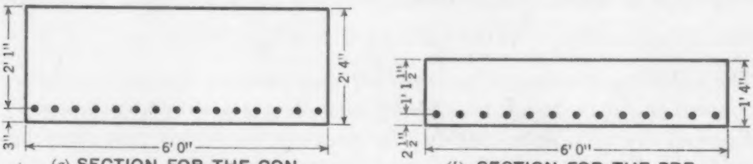
FIG. 4.—TRUCK LOADING FOR MAXIMUM BENDING MOMENT

plus 25% for impact, on an element 1 ft wide, is $M_L = 4\,166\, L \left(0.5 - \frac{1.4}{L}\right)$. If the length, L, equals 27.33 ft, $M_L = 22\,800$ ft-lb; $f_{ac} = 650$ lb per sq in.; $f_{as} = 18\,000$ lb per sq in.; and $n = 15$. In current design practice, $p_b = 0.065$, and the thickness of the slab is found to be $d = 0.099\sqrt{38\,600 + 22\,800} + 3.0 = 27.6$, or 28 in.

The next step is to design the pre-stressed slab. In Fig. 5(*b*) let $d = 16$, $p_b = 0.0086$, and $f_{as} = 45\,000$ lb per sq in. The effective span length, L, is $25 + 1.33 = 26.33$ ft; the dead load is $1.33 \times 150 + 65 = 265$ lb per sq ft; and the bending moments are computed to be, respectively, $M_D = 22\,900$ ft-lb; and $M_L = 21\,700$ ft-lb. Since there is no top reinforcement, $p_t = 0$ and $q_t = 0$. Furthermore, $q_b = \frac{13.5}{16} = 0.844$; and by Equation (3), $k = 0.537$ and C (in

[13] "What the D. L. & W. R. R. Is Doing in Concrete Design", by M. Hirschthal, M. Am. Soc. C. E., *Railway Review*, October 9 and 16, 1926.

Equation (4)) equals $0.0833 + 0.037^2 + 14 \times 0.0086 \times (0.844 - 0.537)^2 = 0.096$. By Equations (5), (6), and (7), the stresses are computed to be (in pounds per square inch): $f_{tD} = 500$; $f_{bD} = -430$; $f_{sD} = -4\,290$; $f_{tL} = +473$; $f_{bL} = -408$; and, $f_{sL} = -4\,060$. By Equations (12) to (16),



(a) SECTION FOR THE CONVENTIONAL DESIGN

(b) SECTION FOR THE PRE-STRESSED DESIGN

FIG. 5.—COMPARISON BETWEEN THE CONVENTIONAL AND THE PRE-STRESSED DESIGNS

the stresses in the slab due to introducing preliminary stress in the steel are: $D_1 = 0.9914$; $k_1 = 0.497$; $C_1 = 0.0843$; $f_{tp} = -0.00885\, f_{ps}$; and $f_{bp} = -0.0265\, f_{ps}$. These five values must coincide with Conditions (*A*), (*B*), (*C*), (*D*), and (*E*), and the best method of checking the selected section is, first, to determine, from Conditions (*C*) and (*D*), what "designed pre-stress" in the reinforcement would impose the required stress in the concrete under full working load; and then, by Condition (*E*), to make sure that this preliminary stress is not excessive for steel. By Condition (*C*): $F_t'' = +0.00885\, f_{ps} + 500 + 473 \leq 650$, and, therefore, $f_{ps} \leq -36\,500$ lb per sq in.; by Condition (*D*): $F_b'' = -0.0265\, f_{ps} - 430 - 408 \geq 0$, and, therefore, $f_{ps} \leq -31\,600$ lb per sq in.; and, by Condition (*E*): $-F_s'' = -(f_{ps} - 4\,290 - 4\,060) \leq +45\,000$, and, therefore, $f_{ps} \geq -36\,650$ lb per sq in. The similarity between the optimum values of the "designed pre-stress" to be imposed in the reinforcement, in order to utilize concrete (see Condition (*C*)) and steel (Condition (*E*)) to the best advantage, indicates that the section is well balanced in this respect. Conditions (*A*) and (*B*), for dead load only, will be usually satisfied if Conditions (*C*) and (*D*), for full loading, are satisfied. Nevertheless, all these conditions should be checked independently.

From Conditions (*C*) and (*E*) it is possible to state that $-36\,650 \leq f_{ps} \leq -36\,500$ lb per sq in.; and, assuming that $f_{ps} = -36\,500$ lb per sq in., the stresses due to dead load are (see Conditions (*A*) and (*B*)): $F_t' = +0.00885 \times (-36\,500) + 500 = +177$ lb per sq in.; and $F_b' = -0.0265 \times (-36\,500) - 430 = +538$ lb per sq in. It is unnecessary to compute the value of F_s' because the reinforcement is always stressed more under full loading.

Under maximum live load the corresponding stresses are: $F_t'' = +177 + 473 = 650$ lb per sq in.; $F_b'' = +538 - 408 = +130$ lb per sq in.; and $F_s'' = -36\,500 - 4\,290 - 4\,060 = -44\,850$ lb per sq in.

It will be noted that the design is especially well balanced. Tension can never occur in the concrete and the compression stresses are a minimum (+ 177 lb per sq in. at the top, and + 130 lb per sq in. at the bottom). This margin should be sufficient to absorb any accidental inaccuracies introduced by computations or assumptions, before tension stresses would appear in the concrete. Before the live load is applied, the reinforcement is under a tension of 40 790 lb per sq in. The live load adds only 4 060 lb per sq in., an unimportant increase of 10 per cent.

The advantages of pre-stressing, compared with the conventional design, are demonstrated in Table 1. The analysis indicates that the depth of the

TABLE 1.—ADVANTAGES OF PRELIMINARY STRESSES

Description	Conventional design	Pre-stressed design	Percentage, decrease
Cross-Sectional Areas:			
Concrete, in square feet	2.33	1.33	41.5
Steel, in square inches	2.12	1.65	22.0
Stresses, in Pounds per Square Inch:			
Under Dead Load Only:			
At the top	+409	+177	....
At the bottom	*	+538	....
In the steel		−40 290	....
Under Full Load:	−11 300		
At the top	+650	+650	....
At the bottom	*	+130	....
In the steel	−18000	−44 850	....
Percentage increase in steel stress under live load	59	10	....

* Tension cracks.

slab should be increased from the center toward the supports, where it reaches a maximum value of $d_0 = \frac{3}{2}\, q_b\, d = \frac{3}{2} \times 13.5 = 20.25$ in. At the supports the stresses in the concrete are zero at the top and, at the bottom:

$$-\,2\,\frac{P}{A} = \frac{2\, f_{ps}\, p_b\, bd}{b\, d_0} = 2 \times 36\,500 \times 0.0086 \times \frac{16}{20.25} = 498 \text{ lb per sq in.}$$

Instead of computing the theoretical curve of the bottom of the slab, or beam, it is simpler to design sections for several points along the span. In the present case (see Fig. 5), the depths are found to be as follows:

x, as a ratio of:	d, in inches
$0.05\,l$	19.0
$0.1\,l$	17.8
$0.2\,l$	16.9

The total weight of the pre-stressed slab unit is about 19 tons, 37% less than for the conventional design.

Example 2.—Consider the same pre-cast slab (Fig. 5*b*) as in Example 1, except that it has double reinforcement. Let $d = 13$ in.; $p_t = 0.012$; and $p_b = 0.0105$. The dead load is $1.08 \times 150 + 65 = 227$ lb per sq ft; $M_D = \frac{1}{8} \times 227 \times 26.08^2 = 19.200$ lb-ft; and $M_L = 4\,166 \times 26.08 \left(0.5 - \frac{1.4}{26.08}\right)^2 = 21.500$ lb-ft. By Equations (3) and (4), $k = 0.495$ and $C = 0.1208$; and by Equations (5), (6), and (7), $f_t = +\,0.0242\,M$; $f_b = -\,0.0247\,M$; and $f_s = -\,0.258\,M$.

Stresses due to external loads are (in pounds per square inch) computed as: $f_{tD} = +\,465$; $f_{bD} = -\,475$; $f_{sD} = -\,4\,950$; $f_{tL} = +\,520$; $f_{bL} = -\,531$; and $f_{sL} = -\,5\,550$. By Equations (8), (9), and (10), $K_1 = 1.157$, $k_1 = 0.447$, and $C_1 = 0.0988$. By Equations (12) and (13), the pre-stresses are: $f_{pt} = +\,0.00993\, f_{sp}$ and $f_{pb} = -\,0.0325\, f_{sp}$. On the other hand, the pre-stress

in the steel required by Conditions (C), (D), and (E) (in pounds per square inch), is:

$$F_t'' = +\,0.00993\,f_{sp} + 465 + 520 \leqq +\,650; \quad \text{or,} \quad f_{sp} \leqq -\,33\,700$$

$$F_b'' = -\,0.0325\,f_{sp} - 475 - 531 \geqq 0; \quad \text{or,} \quad f_{sp} \leqq -\,32\,700$$

and

$$F_s'' = -\,(f_{sp} - 4\,950 - 5\,550) \leqq +\,45\,000; \quad \text{or,} \quad f_{sp} \geqq -\,34\,500$$

which indicates that f may be selected between $-$ 33 700 and $-$ 34 500 lb per sq in. Assuming that $f_{sp} = -\,34\,000$ lb per sq in.:

$$F_t' = +\,0.00993 \times (-\,34\,000) + 465 = +\,127$$

and,

$$F_b' = -\,0.0325 \times (-\,34\,000) - 475 = +\,631$$

For full loading:

$$F_t'' = +\,127 + 520 = +\,647 \text{ lb per sq in.}$$

$$F_b'' = +\,631 - 531 = +\,100 \text{ lb per sq in.}$$

and,

$$F_s'' = -\,34\,000 - 4\,950 - 5\,550 = -\,44\,500$$

Thus, all conditions are satisfied. The cross-section of concrete required is only 1.08 sq ft which is less than 50% of the area required by conventional design. The cross-section of steel is 3.51 sq in. which is 66% greater than conventional design. Such a design may be recommended only if the thickness or the weight of the bridge must be decreased in order to meet local conditions.

The slabs can be made even thinner by introducing a concrete of higher strength; for example, if the allowable stress in the concrete can be increased to 1 500 lb per sq in., the pre-stressed slab in Problem 1 will be 10 in. thick, with a steel ratio of $d = 0.021$.

Tests on Prestressed Concrete

To test the practicability of introducing preliminary stresses in reinforced concrete, E. Freyssinet constructed beams reinforced symmetrically with different kinds of steel bars and wires, including piano wire with an elastic limit of 284 000 lb per sq in. Initial tensile forces were applied such as to produce stresses from 99 600 to 256 000 lb per sq in. Mortar was placed in small portions and vibrated; samples were tested by bending; and the results were very favorable. Rupture occurred at loads much greater than were theoretically computed. In some cases the samples failed in the steel and, in some cases, in the concrete; but in the latter cases, too, it was the steel that produced the rupture by exceeding its yield-point stress. The deflections were small, being almost always only about one-fifth of the deflection observed in conventional design. What is of a special importance as applied to reinforced concrete, the deformations were completely reversible. No cracks appeared until tensile stresses due to working loads exceeded the initial compression stresses by from 280 to 430 lb per sq in.

In 1935, E. Freyssinet conducted reversed bending tests[14] on a post 40 ft long, fixed for a length of 6 ft 7 in. from the base. Repeated loads were applied from

[14] *Memoires* de la Société des Ingénieurs Civils de France, September-October, 1935.

+ 1 000 to − 1 000 lb at the rate of eight times per minute. A controlling post of the conventional design, heavily reinforced with 286 lb of steel, was fissured after a few hundred deflections and broke after a few thousand deflections. A "prestressed" post of the "treated" concrete with only 110 lb of steel was subjected to 500 000 deflections without any alteration.

In 1934, W. K. Hatt, M. Am. Soc. C. E., tested a series of 8 by 14 in. beams, 13.5 ft long, reinforced with two rods 0.816 in. in diameter. These beams were loaded to produce bending moments from zero to 260 000 in-lb.[15] The bond

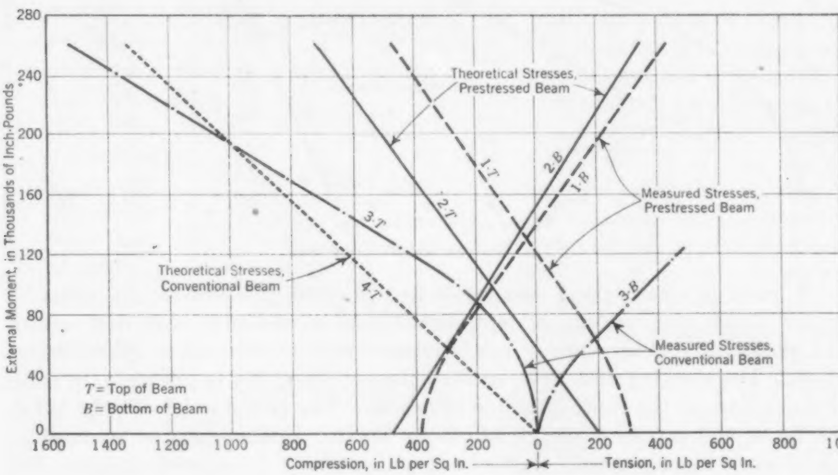


FIG. 6.—UNIT CONCRETE STRESSES

between the rod and the concrete had been destroyed with an asphaltic coating and, after setting, the reinforcement was prestressed to loads of 8 372, 11 151, and 15 273 lb per sq in. Stresses in the concrete and in the steel were measured exactly. A beam of the same section, but not prestressed, was also tested.

Figs. 6 and 7 show the unit stresses at the top and at the bottom of the concrete and in the steel: (1) Measured exactly, in tested prestressed beams; (2) computed by the theory of the prestressed beam; and (3) measured exactly, in a tested conventional beam. The computed stresses at the bottom of the concrete agree well with tests. Some greater difference is observed for the top of the concrete where the actual effect of prestressing proved to be greater than the computated value. Measured deflections follow strictly theoretical conclusions. The prestressing first produced upward bending. Under the bending moment of 100 000 in-lb, the beam regained its initial form. The conventional beam deflected under the same load as much as 0.035 in. Then the load was increased so as to produce a unit stress of 1 050 lb per sq in. in the concrete in the conventional beam; and the deflection measured in the conventional beam was 4.4 times that in the prestressed beam.

[15] *Engineering News-Record*, September 13, 1934.

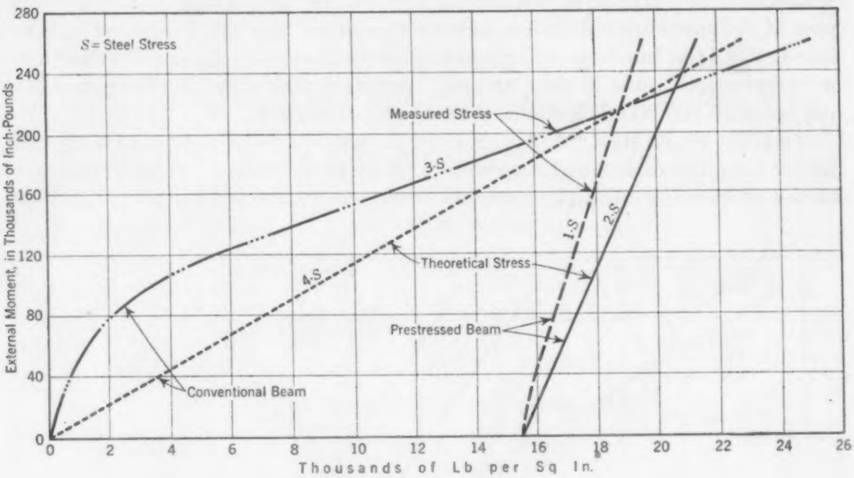


FIG. 7.—UNIT STEEL STRESSES

A most interesting test was conducted in 1936, in Germany.[16] A model girder, 65.5 ft long, varying in depth between 33 in. and 46 in. was used. Both the horizontal and the vertical reinforcement were prestressed to 70 000 lb per sq in. The working unit stress in the concrete was 2 056 lb per sq in. in compression and, at the steel, 78 000 lb per sq in. The girder weighed only 150 lb per lin ft, and carried a load of 940 lb per lin ft. Under repeated loading the girder acted like a perfectly elastic body and disclosed no signs of fatigue. Reinforced with steel having an elastic limit of 140 000 lb per sq in., the girder had a factor of safety of 2 in the steel and of about 4 in the concrete. The deflection was unusually small.

The details of this experiment have not been made public, but, on the basis of this test, it is concluded that a girder of 327 ft span, with an average depth equal to 0.05 L, could be built economically. Such a girder would carry 600 tons of its own weight and 300 tons of useful load. The deflection would be only one-third of that in a steel structure of the same dimensions.

METHODS OF PRESTRESSING THE REINFORCEMENT

Three methods can be used to prestress the material required:

(1) Mechanical devices of various forms can be devised. Ends of the reinforcement bars can be hooked and connected to the special anchor-beams by means of these hooks, intermediate hooked bars and clamps being fixed on the anchors. One of the anchor-beams is subjected to the action of jacks held against a strong abutment. In this manner the reinforcement bars are stretched to a predetermined value. Devices of this type are suitable for the manufacturing of precast structures in a concrete yard where supports can be built permanently.

For field work, it may be preferable to use the forms as supports, strengthening them with end blocks and struts and carrying loads from one end of the

[16] *Concrete and Constructional Engineering*, April, 1936.

form to the other. In this case, jacks are placed between the anchor-beam and the end block.

(2) Reinforcement can be easily prestressed by heating the rods. To achieve the expansion corresponding to the stress of 90 000 lb per sq in., it is required to introduce a temperature 260° C above the normal temperature. The problem of applying heat is simplified by electrical devices, as has been demonstrated by M. Ruml, in Belgium, who applies this process to prestressing pipes.

However, the heat-treatment method should be tested carefully. In this process the steel is subjected to high temperature, and placed in contact with concrete while the latter is hardening, with the result that the surrounding concrete is heated. In many cases, heated concrete has proved to be unreliable. E. Freyssinet, who introduced the heating of the concrete as an essential element of his new method of "treated" concrete, acknowledges this fact but explains it as being due to non-uniform heating. On the other hand, the other noted French engineer, M. Lossier,[17] cites the precast electric insulators produced in 1913 at 185° F, when the concrete was rammed. At the end of four years they were in an excellent condition, but three years later their strength was reduced much below that of ordinary insulators. It appears possible that initial heating is injurious to chemical actions over long periods of time. If it results in destroying the bond between the steel and the concrete even "in the long run," heat cannot be used for prestressing.

(3) An ingenious method of "self-prestressing" was used by E. Freyssinet for piles in combination with his "treated" concrete. Piles were manufactured in place in sections between two cylindrical moulds. The external mould consisted of half-collars which were fixed with screw clamps. The internal mould is made of a steel tube surrounded by an envelope of india rubber with a cotton armor. The space between moulds is closed by an annular plate with holes for rods and for placing the concrete, and rods are clamped on the top of the plate.

The concrete is poured between the moulds and vibrated by external vibrators. The excess water escapes through joints between the collars of the external mould. Then the vibration is stopped and a hydraulic pressure of 285 lb per sq in. is exerted in the internal mould between the steel tube and the rubber envelope. This pressure is transmitted to the concrete and through it to the top closing plate because, immediately after vibration, the concrete acts as a liquid. The plate rises and stretches the rods that are fixed to its top, almost to their elastic limit. The pressure is maintained for 20 min, the water leaks freely through the joints, and the reinforcement is prestressed as required.

Conclusions

From studies made by the writer and described in this paper, six broad conclusions seem to be justified, as follows:

1.—In reinforced concrete design, there are great advantages in increasing or decreasing, artificially, the stresses in one or the other materials involved. One of the methods consists in introducing preliminary tension in the reinforcement and tests by E. Freyssinet and others show that this method is practicable.

[17] *Concrete and Constructional Engineering*, August, 1936.

2.—Invariably the results of introducing pre-stressed reinforcement in beams and girders are favorable when the preliminary stresses can be designed so as to counterbalance the stresses produced by dead loads and live loads.

3.—Pre-stressed designs may result in exclusively great savings of material or they may permit considerable increases in maximum span length. In either case, the structures are more reliable than in the case of conventional design because the tensile stresses in concrete are completely canceled and the compressive stresses that occur over the entire cross-section tend to prevent the occurrence of cracks.

4.—The arrangement of the reinforcement is simplified considerably because the diagonal tension is appreciably less than in the conventional design, and in continuous beams, reinforcement is not necessary at the top.

5.—In order to gain the greatest advantages from the principle of prestressing, it is important that the reinforcement have a high yield-point stress and that special devices for applying the preliminary stresses be available. The cost involved in both cases is not serious.

6.—The effect of introducing the pre-stressed design into reinforced concrete will be to develop much lighter, more reliable, cheaper, and simpler structures, thus increasing the field of usefulness of this material considerably.

APPENDIX

Notation

The following symbols, which conform to the Symbols for Mechanics, Structural Engineering, and Testing Materials[18] approved by the American Standards Association in 1932, are offered for the guidance of readers and discussers of this paper:

A = effective area; A_1 = effective area that resists prestresses.

a = a subscript denoting "allowable".

b = breadth, or width, of a beam; as a subscript, b denotes "bottom".

C = a constant.

c = a subscript denoting "concrete".

D = dead load; as a subscript, D, denotes "dead load".

d = depth of a beam; dx = depth, d, at a distance, x, from the left support,

E = modulus of elasticity.

f = net unit stress; f_c = unit stress in concrete; f_s = unit stress in steel; $+ f_p$ = unit prestress (initial compression); $- f_p$ = unit prestress (initial tension); f_D = dead load stress; f_t = unit stress at the top of a beam; f_b = unit stress at the bottom of a beam; f_a = allowable unit stress; f_L = unit stress due to live load.

I = moment of inertia of the effective area; I_1 = moment of inertia that resists prestresses.

j = ratio of lever arm of the resisting couple, to the distance between the outer compressive fiber and the point of application of the resultant tensile stress.

[18] A. S. A.—Z10a—1932.

K = a substitution factor = $1 + (n - 1)(p_t + p_b)$; K' = the substitution factor for no top reinforcement = $1 - p_b$.

k = ratio of distance from the neutral axis to the top fiber of a reinforced concrete beam, to the depth of the beam.

L = live load; as a subscript, L denotes "live load".

l = length; as a subscript, l denotes "longitudinal" stress.

M = moment.

n = ratio, $\frac{E_s}{E_c}$.

P = a concentrated load; a "designed prestress" force applied to a member.

p = percentage of steel in the entire cross-section of a beam; p_t = referred to the top steel; p_b = referred to the bottom steel.

q = ratio of distance from the center of the reinforcement to the top fiber of the beam, to the depth of the beam.

s = a subscript, denoting "steel".

t = a subscript, denoting "top".

u = unit bond stress.

V = total shear.

v = unit shear stress.

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PAPERS

PRACTICAL APPLICATION OF SOIL MECHANICS

A SYMPOSIUM

LEVEES IN THE LOWER MISSISSIPPI VALLEY

By Spencer J. Buchanan,[1] Assoc. M. Am. Soc. C. E.

Synopsis

A levee may be defined as an earth embankment constructed along the margin of a stream to restrain its flow within a desired course during flood stages. The system of levees, as it exists to-day in the Lower Mississippi Valley, is one of Man's greatest works, involving approximately 761 000 000 cu yd of material throughout its length of 1 615 miles. The type used is similar in many respects to modern earth dams. In fact, it is natural that there should be a close similarity because a levee is merely an earth dam of great length, designed to function during relatively short intervals, after long lapses of time. This paper is concerned with the general features considered in the design of a levee and the use of the relatively new tool, Soil Mechanics, in solving unusual problems arising in connection with levee design.

Introduction

The first levee on record, to be constructed in the Lower Mississippi Valley, was built in the vicinity of New Orleans, La., by De la Tour under the orders of the founder of the city, Sieur de Bienville. The date of this work was between 1717 and 1727. It was a modest start as compared to the present system, but it proved so successful that, in 1743, the French Colonial Government ordered all land owners in that vicinity to "follow suit" and gave them one year to complete their units under penalty of forfeiture of their holdings immediately to the Crown. With the development of the Valley grew a haphazard system of levees which provided uncertain protection from the floods of the river. The resultant confusion, together with growth of national interest in the matter, resulted in the creation of the Mississippi River Commission by an Act of Congress in 1879, for the co-ordination of the efforts of the local interests and for the administration of the work of the National Government in controlling and improving the river.

The growth of the system under the administration of this Commission has been tremendous and its efforts have been marked with success as indicated by the satisfactory confinement of the 1929 and 1937 floods. The extent of the system at present is shown by Fig. 1. Accompanying this growth there has been a corresponding reduction in the area of the floodway and natural storage basins; greater flood stages have resulted, requiring higher levees. The average height at this time is approximately 22 ft. However, this height is as great as 50 ft at isolated points where the line crosses lakes or bayous. As the growth in size progressed, the experience of those directing the work broadened. The weaknesses in design became apparent under actual flood conditions and sufficient revision was made to insure stability.

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During the past few years the methods of levee construction have changed, the value of property protected has been enhanced, and the demand for designs complying with strict engineering practices has increased. The results of these changes have been many. For example, the change in construction methods makes it possible to complete the new and larger levees in a fraction of the time formerly required with mules and wagons or with men and wheel-barrows. Consequently, the foundations are loaded more rapidly and with heavier loads, and the chances of their failure are increased. When the circumstances are such that stable foundations are evident, and material is available with which sufficient experience has been accumulated, designs based on experience have been adequate. However, when the limits of the location are restricted, and it is necessary to construct a unit across an old lake bed or on some poor foundation, then the use of the new tool, Soil Mechanics, serves a need of long standing. The use of this branch of civil engineering permits design to be founded on a strength-*versus*-stress basis.

FIG. 1.—LEVEE SYSTEM, MISSISSIPPI RIVER, 1936

Essential Features of Design

The ultimate design of a levee requires consideration of the following essential features: Side slopes, foundation, and control of seepage. The first two of these features are of the utmost importance because, more than anything else, the structures must be stable for any conditions. The control of seepage is important and justifies discussion because of its effect on the stability of the structure. However, the volume of water lost is of little importance as its conservation is rarely desired in a flood control system. Furthermore, the normal drainage system of the areas adjacent to levees is generally more than adequate for the disposal of water accumulating through seepage.

Side Slopes.—The side slopes adopted for the standard sections of the levees

over the period from 1882 to date, are shown by Fig. 2. The elevation of the maximum probable flood is that contemplated by the Flood Control Project authorized by Act of May, 1928. It may be noted that, as the size of the structures has increased, there has been a gradual reduction of the inclination of the land-side slope. The river-side slope has remained practically unchanged from the beginning. The broken land-side slope, as indicated for the banquette sections used between 1896 and 1914, was found to be undesirable in certain

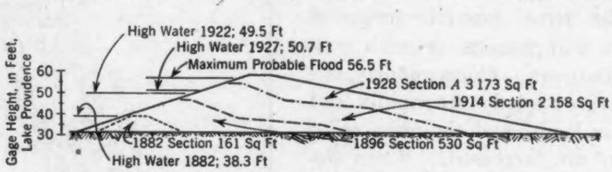

FIG. 2.—STANDARD LEVEE SECTIONS, LOWER MISSISSIPPI RIVER

respects for the high levees on the Lower River. The steep land-side slope above the banquette proved unstable in some instances when confining the maximum floods for which they were designed. This unstable condition was probably due to the hydrodynamic forces created by the seepage, acting on the stepped slopes. In many instances slides or general sloughing actually occurred. For the Lower River, the levee section was altered to give a straight land-side slope.

The slopes in use at present vary somewhat with the material available in the borrow-pit areas. Table 1 shows this variation for the basic materials, sand, silt, and clay.

TABLE 1.—SLOPES FOR CLAY, SILT, AND SAND

Section	Predominating material	Crown width, in feet	River-side slope*	Land-side slope* containing assumed topmost flow line of following slope
A	Clay (75%)	10	1 on 3.0	1 on 6.0
B	Silt	10	1 on 3.5	1 on 6.5
C	Sand (75%)	12	1 on 5.0	1 on 8.0

* Topmost flow line assumed to spring from river-side slope, 1 ft below crown of structure.

The stability of these standard sections has been investigated, by the method described subsequently, and for normal conditions was found to vary between 1.5 to 3.5. This factor of safety is considerably greater than the average for earth structures of the levee type. The *B* section (Table 1) predominates throughout the Mississippi Valley. It has been found that the use of clay, of the richness requiring the *A* section, is generally undesirable. Consequently, the *A* section is rarely used. The foregoing sections have been developed over a long period of time and are based upon experience gained with the soil existing in the Valley. They should not be considered applicable to other localities where the material available may be entirely different although bearing the same local classification.

For situations in which new levees necessitate the use of material with which there has been little experience, the application of soil mechanics makes it possible to develop a levee design, based on the strength of the material, to give

any factor of safety desired. The method of design depends on the nature of the material, on whether it is cohesionless, as a sand, or cohesive, as a silt and clay. For the slopes of structures formed of sands or like materials, the inclination must not be greater than the angle of internal friction. Their failure is by sloughing along the surface of the slope when this criterion is exceeded rather than in the manner described subsequently that applies to cohesive silts and clays. The stability of slopes formed of cohesive materials depends upon the shearing strength of the material along a deep-seated cylindrical surface. This cylindrical surface generally extends from a point just back of the crown of the slope, down through the section, and through the toe of the structure. The segment of the slope lying above this arc, in failing, appears to slide downward, rotating about the center of the arc. The method of locating the approximate position of this center of rotation for various slopes as established by W. Fellenius[2] and amplified by Charles Terzaghi,[3] M. Am. Soc. C. E., has been found to be satisfactory. Evidence of this type of failure in shear due to the over-stressing of the material has been observed in innumerable cases, particularly in levees constructed of material too wet for use.

The steps taken in the design of a slope, neglecting the effect of seepage on its stability, are relatively simple and much in accord with the prevailing practice. They are as follows: (1) The approximate position of the center of rotation for the arc of failure is determined; (2) the part of the slope of unit thickness, above this surface, determined by the arc of failure, is subdivided into vertical segments (the number—usually ten or twelve—depends upon the size of the structure); and (3) the gravitational force acting on each segment is determined and resolved into its normal and tangential components acting along the arc at the vertical projection of the center of gravity of the segment on the arc of failure.

Summation of the tangential components gives the force, T, tending to produce displacement. The force resisting this tendency is the summation of the frictional and cohesive forces acting along the arc. The total frictional force is determined by adding the normal components and multiplying the sum, N, by the tangent of the angle of internal friction, ϕ. The cohesive force is determined by multiplying the length of the arc, l, by the unit cohesive strength, c, of the material. The cohesive strength is obtained by a shear test performed upon a specimen of the material, prepared in a condition to simulate the most adverse situation expected. Cohesive strength is considered to be entirely independent of the pressure, depending solely upon the material and its condition. Thus, the stability of a slope or its factor of safety against sliding, may be determined by the expression:

$$\text{Factor of safety} = \frac{N \tan \phi + c\, l}{T} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

The foregoing procedure is repeated until the center of rotation and radius of curvature are found, that give a minimum value of the factor of safety for the

[2] "Jordstatiska beräkningar med friktion och kohesion för cirkulär cylindriska glidytor", W. Fellenius, Kungl. Väg- och Vallenbyggnadskårens 75-årsskrift, pp. 79–127, Stockholm, 1926.

[3] "The Mechanics of Shear Failures on Clay Slopes and the Creep of Retaining Walls", Charles Terzaghi, *Public Roads*, Vol. 10, No. 10, December, 1929, pp. 177–192.

slope in question. The corresponding arc is then accepted as the critical surface upon which failure is most likely to occur. Several slopes are analyzed by this method and the one having the desired factor is selected.

A factor of unity for the stability of the side slopes indicates a balance of the forces. However, a slight deviation of less than unity can not be taken as a direct indication that failure is imminent. Values of less than 0.75 may be taken as indicative of an unsatisfactory situation and values greater than 1.25 can be taken as assurance of stability. Although this method of analysis may have its limitations, it does provide an excellent yardstick whereby analysis may be made of a situation about which engineers have had previously only rule-of-thumb methods to serve as guides.

Another element enters into the stability of the land-side slope after seepage through the structure has developed, or for the river-side slope following the passing of a flood and the removal of the impounded water. Failures have been observed of the river-side slopes of levees under just such conditions. The hydrodynamic force created by the seepage water, either flowing into or out of the section, produces an overturning moment in the part of the slope lying above the arc of failure. For example, considering a levee through which flow has fully developed, the flow and equipotential lines being arranged as shown by

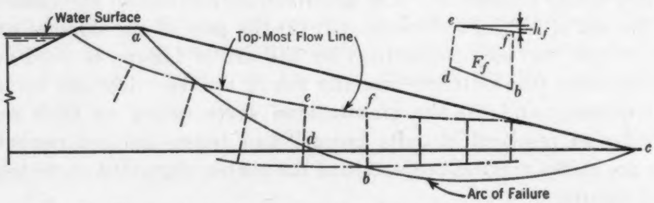

FIG. 3.—HYDRODYNAMIC EFFECT OF SEEPAGE ON STABILITY OF SLOPES

Fig. 3, the loss of head, h_f, occurring through the part above the arc of failure, $b\,d\,e\,f$, is due to the friction overcome by this seepage water in flowing through the material. The net effect of this loss of head is considered as a force acting in the general direction of the tangential component tending to produce failure and should rightfully be considered in such an analysis. (In Fig. 3, F_f is the force tending to cause displacement in the direction of flow, equal to h_f times unit weight of water times area surface, fb, of unit thickness.

Although further research must be completed before definite conclusions may be safely drawn, a tentative method of analysis considering the effect of seepage water upon the stability of side slopes was advanced in a memorandum of the U. S. Waterways Experiment Station[4] this year. This analysis is similar to that reported by E. Meyer-Peter[5] and concurred in by D. W. Taylor, Assoc. M. Am. Soc. C. E., in a paper to be published in the early fall. A paper on this hydrodynamic effect is to be prepared soon; hence it is only mentioned here.

Foundations.—It has been stated that the prime requisite of a levee was stability under all conditions. After the stability of the side slopes has been

[4] A Report of Experiments and Investigations to Determine the Efficacy of Sub-Levees and Berms in the Control of Seepage, *Technical Memorandum No. 101-2*, July 1, 1937.

[5] "Beitrag zur Berechnung der Standsicherheit von Erddämmen", von E. Meyer-Peter, H. Favre, and R. Müller, *Schweizerische Bauzeitung*, Vol. 108, No. 4, July 25, 1936, pp. 35–37.

determined, the next essential feature to be considered is the foundation. It is not unusual that a setback unit of a levee, required by some channel change or improvement such as a cut-off, is constructed in one season or during a few months just prior to the annual flood period. It is then that a reliable method of design to assure stability is fully appreciated.

In the past, one method[6] of dealing with a weak foundation where the lateral flow of an unconsolidated stratum of plastic material could be expected, was to build up the section and await the anticipated failure. After failure occurred, the placement of dirt was renewed, and the process continued until all the soft material was either squeezed out or until the levee had become stable for other reasons. This procedure required time and plenty of resources. An empirical method was developed about 1933 for the design of berms, which proved to be a very good remedy for this problem. This work was done by T. A. Middlebrooks, Assoc. M. Am. Soc. C. E., and was based entirely upon experience with such problems. His method of analysis had its limitations and has been supplanted by one described in the following paragraph.

In this case, again, the methods developed in the field of soil mechanics can be brought into play and the structures designed so that the stresses created by the structure will not exceed the strength of the foundations. The method of analysis contributed by Leo Jürgenson[7] makes it possible to compute the stresses created by this type of structure. Either the quick direct shear test, or the squeeze test, on specimens from undisturbed samples, can be used to obtain the ultimate strength of the material *in situ*. With these data it is a relatively simple matter to determine the stability of a foundation or the necessary adjustment of the design to give the factor of safety desired.

An application of this method of analysis to three full-scale levee structures is interesting and worthy of mention. This investigation was made in May, 1935, for the purpose of corroborating methods used in the design of the foundations of numerous earth dams in the Muskingum Valley, in Ohio. Of the three units, founded on similar material, one had been found stable and the other two had experienced complete failure, due to the over-stressing of their foundations. The first of these units was constructed after 1930 and the latter two prior to that date. The results of this application are illustrated in Table 2.

The evidence indicated by the analysis of the first two units appears to justify confidence in this method. Although that for the third, Milliken Bend, unit appears to fail in proving the desired point, it was the only one of the group whose original cross-section departed appreciably from the triangular shape. The analysis applies to triangular cross-sections. A comparison of the computed values of shear with those measured in models by photo-elasticity shows a variation of 1%, 2%, and 30% for the units in the order shown (Table 2). Thus, it is indicated that this method of analysis has its limitations when applied to a section deviating from the true triangular shape. Further use of this method of analysis on other projects has been made since 1935 and to date (1937) no difficulties have arisen. The structures have been satisfactory in every respect when the basic assumptions have been met.

[6] "The Improvement of the Lower Mississippi River for Flood Control and Navigation", by D. O. Elliot, *Bulletin*, U. S. Waterways Experiment Station, May 1, 1932, Vol. 2, p. 177.

[7] "The Application of Theories of Elasticity and Plasticity to Foundation Problems," by Dr. Leo Jürgenson, *Journal*, Boston Soc. of Civ. Engrs., Vol. XXI, No. 3, July, 1934, pp. 206–241.

Settlement is an important factor in levee construction, as it is in building construction. However, the effect is quite different, for little damage, if any, is experienced by a levee through its settlement. The principal benefit from the predetermination of this factor is in the economy effected. A reliable estimate permits the gross grade to be adjusted at the time of initial construction to allow for the settlement. Thus, no subsequent enlargements are necessary to maintain the desired net section or grade. It has not been unusual for these structures when placed across old lake beds or bad swampy areas to experience 3 to 5 ft of settlement. Of course, allowance is made at the time of construction for the normal shrinkage of the fill. However, when in addition to this shrinkage a settlement occurs, the section must be restored. The cost per unit of fill for the enlargements is naturally greater than the initial unit cost. In addition, emergency protection may be required during a major flood. This emergency protection is in the form of sand bags or mud boxes placed in depressions caused by settlement, and is very expensive.

TABLE 2.—SAFETY FACTORS FOR THREE LEVEES

Item No.	Name of unit	Basic Dimensions, in Feet			Increase in pressure caused by structure, in tons per square foot	Maximum stress, in tons per square foot	Measured strength, in tons per square foot	Factor of safety
		Height	Base width	Depth of underlying soft stratum				
1	Halpino Levee*	36	432	27.0	1.8	0.113	0.152	1.34
2	Duckport Levee	30	270†	32.5	1.5	0.183	0.165	0.90†
3	Milliken Bend Levee	37	270	27.0	1.8	0.313	0.163	0.53‡

* No difficulty was experienced with this levee.

† After this levee failed the section was successfully reconstructed to a new base width of 347 ft, changing the factor of safety to 1.16.

‡ Structure failed, as was to be expected. The structure was successfully reconstructed to a new base width of 416 ft, changing the factor of safety to 0.80.

The detection of portions of a new unit in which excessive settlement may be expected is not a difficult matter. The site of a proposed unit as well as the borrow-pit areas are explored in a routine manner by borings, prior to the completion of the initial design. Adequate samples are taken during this exploration and subjected to tests as described later in this paper to check the general condition of the foundation, and to detect both the extent and depth of strata of cohesive material containing an apparent excess of water. Analysis of the evident weak spots by the Terzaghi theory of consolidation, combined with data from the consolidation tests of a few well-selected undisturbed samples, is sufficient to form the basis of an adjusted gross grade. The method for determining the settlement of a levee is illustrated subsequently in the solution of a typical problem.

Control of Seepage.—Seepage through the levees in the Lower Mississippi Valley, and its control, have formed the bases for the design of the standard sections previously described. It has been assumed, on the basis of past experience, that the topmost flow line of seepage through a section traveled on slopes of 1 on 6 to 1 on 8, depending on the material. The position of the point of entrance on the river side is fixed by the free-board desired, usually 1 ft of free-board being allowed for the super-flood anticipated. It was further as-

sumed that the seepage through the structures would develop fully during the flood period. In so far as the records to the present time show, the duration of this period has not exceeded 120 days, while, of course, the flood waters would remain at the assumed free-board height for a small fraction, say, 15% of the time.

The shape of the section has been made to contain the topmost flow lines just mentioned. During an inspection of some of the levee units in the vicinity of Memphis, Tenn., the writer observed no evidence of serious seepage coming through the levee section. Although difficulties were experienced with the seepage through the foundations at some locations, this can not be attributed to any weakness in the design of the section. Pervious strata frequently exist in the foundations of levees and are the source of some trouble of local nature. Control of this excessive seepage is achieved by the use of sub-levees, or by placing a ring of sand bags so as to isolate the small local spots. Either of these means of control develops a negative hydrostatic head neutralizing or reducing to a harmless quantity the water leaking through the foundation. Thought is now being devoted to methods of sealing such pervious foundation strata so that this undesirable condition may be remedied.

Problems involving structures founded on pervious material are analyzed with the aid of models constructed in a glass-sided flume. The conditions under which these experiments have been conducted were arranged to simulate various positions of ground-water. The data from these tests are used to estimate both the position of the topmost flow line in, and the quantity of flow through, the section.

Structures founded on impervious strata may now be analyzed by methods such as one developed by Glennon Gilboy,[8] Assoc. M. Am. Soc. C. E. This problem is usually complicated by the fact that no selection of material is exercised in the construction of a levee. Hence, the usual assumption that the structure is a homogeneous mass is not justified. The state to which the material in the structure may be compacted is a variable that will affect the flow through the section. This variable can be judged only by experience gained through observation of structures recently completed; a limited number of observations and tests have been made. The details of this type of analysis are treated in the solution of a typical levee problem outlined subsequently in this paper.

The rate of development of seepage through a section of levee is a matter of importance. The maximum period of flood as estimated previously is 120 days, and it is only for a short part of this time that the maximum flood prevails. The rate of seepage development has been a matter of conjecture. Some interesting data have been obtained through observations of the development of the flow through an earth dam at the United States Waterways Experiment Station, at Vicksburg, Miss. These data throw some light on the subject. The dam is 35 ft high, with side slopes of 1 on 3.7 for both up-stream and down-stream faces. It is constructed of loess, a remarkably homogeneous material typical of the Vicksburg area, and is founded upon the same material. The loess, while unlike in some respects to the alluvial soils in the Valley, has, nevertheless,

[8] "Hydraulic-Fill Dams," by Glennon Gilboy, 1st Congress of Large Dams, Vol. 4, Rept. No. 46, pp. 231–267.

permeability characteristics similar to a great proportion of the material used in the levees, that is, the silts requiring the standard *B* section. The material was placed by trucks in layers 6 in. thick, and compacted only by the repeated passage of these vehicles. No exact control was exercised over the compacting operation. The elevation of the impounded water in the reservoir has been maintained at 164.5 ft (mean Gulf level) since May, 1935. The dam was constructed in February and March, 1935, with the spillway crest at Elevation 165.0 (mean Gulf level). A duplicate set of observation wells were placed in the structure at the time of its construction. The locations are shown by

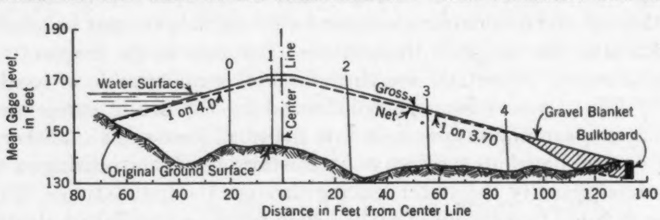

FIG. 4.—LOCATION OF OBSERVATION WELLS, STATION 0 + 66

Fig. 4. Tests showed that the desired compaction was obtained by the constant passage of the trucks over the spread material. The results of the observations are shown by Fig. 5, which indicates that ten months were required for the full development of the seepage through this structure.

The results of these observations show that seepage development is exceedingly slow, and tend to indicate that for sections formed of relatively impervious materials, silts, or finer, the flow does not develop during the flood period.

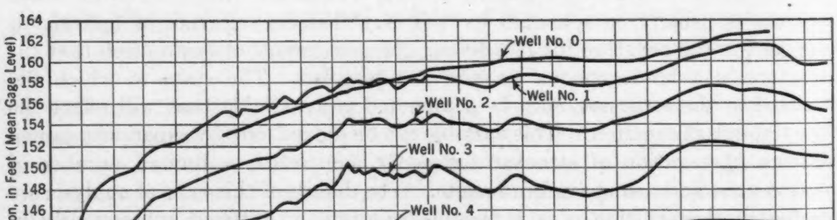

FIG. 5.—DEVELOPMENT OF SEEPAGE THROUGH SECTION 0 + 66

Consequently, sections designed to comply only with the requirements of stability are satisfactory. In the past, numerous instances were reported in which the seepage through the standard banquette section did develop, outcropping at the upper edge of the banquette, and causing damage. The path of flow through the present section is two or three times as long and, consequently, requires a greater time to develop.

For the situation of pervious structures, formed of sand, the seepage may develop during the flood period. However, a unique method for controlling

the seepage through this type of levee has been applied in some instances. This control is provided by means of impervious clay blankets, 3 ft thick, placed on the river-side slope and extended through pervious surface strata into an extensive impervious stratum in the foundation. Information as to the effectiveness of this control is provided by observations made by the U. S. Engineer District Office, in Vicksburg, of the pervious levee closing the gap formed by the famous Mounds Landing Crevasse that occurred during the 1927 flood. These observations were made before and after the placement of the clay blanket, that is, in 1933 and 1937; the crests of the floods of these years were within 0.5 ft of the same elevation at this unit. The seepage, as collected and measured by a system of weirs at the land-side toe for a part of the levee, 920 ft long, was 0.3 cu ft per sec in 1933 and 0.06 cu ft per sec in 1937. The atmospheric conditions during the 1933 flood were not the same as those that existed in 1937; that is, the precipitation in 1937 was the greater. Consequently, part of the measured seepage of 0.06 cu ft per sec may be attributed to drainage of rain water.

The Design of a Levee Unit

The design of levees, as accomplished through the application of soil mechanics, is basically the same as that used for earth dams about which much has been written recently. However, the attack of this problem becomes unique through consideration of several essential features: Location of levee, source of materials, placement of materials, and control of seepage.

Unfavorable locations along a line, as far as foundation conditions are concerned, cannot be avoided as readily as by shifting a dam up stream or down stream a short distance. Deviation from the shortest practicable route, to avoid crossing filled sloughs or lakes, increases the cost of the structures tremendously. Furthermore, the local interests, who are naturally desirous of obtaining protection for a maximum amount of their land, contribute toward the cost of the levees. Consequently, their desires must be given consideration and complied with in so far as is possible. Formerly, to simplify the enormous task when extensive programs have been undertaken, sections, based upon empirical designs, have been standardized with ample allowance for local "sore" spots in the foundations, both as regards unstable conditions and excessively pervious materials. Now, it is possible, through the application of soil mechanics, to alter the design to fit the conditions encountered.

The material adjacent to the river-side toe of the levee must be used for its construction, whether it is sand or clay. The principal reason for its use is the economy effected. The use of the material directly opposite the section minimizes the distance it must be transported, eliminating over-haul. The borrow-pits, being on the river side, are thus located in areas where land values are low and the borrow material is cheap. For the enlargement of sections, this results in even greater savings than for the original construction as the borrow areas have already been adjusted. In general, any attempt toward the selection of material would be fruitless due to the supply, of what is generally reckoned as suitable according to current practice in earth dam design, not being equal to the demand. In some regions, especially the lower part of the river, that at hand is just as suitable as any within about a mile. This situation makes

it necessary to design the structures on the basis of the weakest of available materials. Although this often requires large sections with flat slopes when compared to some dams recently designed and constructed, the problem is simplified and solved at a low cost.

It is important that no borrow-pits or excavations be made adjacent to the levees on the land-side to expose pervious strata extending through to the river side. Exposure of such strata invites trouble from increased under-seepage, certainly; and from destruction of the levee structure due to piping, possibly. Although, in some instances, borrow-pits have been made on the land side, they are the source of never-ending trouble.

Control over the placement of the material in levees, in so far as its condition and state of compaction are concerned, has not been exercised as is the practice in the construction of earth dams. Most of the present system had been completed, or was under construction, at a time when little was known regarding the compaction of fills, antedating 1930. This date (1930) apparently marks the actual beginning of efforts to control compaction. To encourage low bids and costs, it has been necessary to minimize the restrictions on methods of construction. If the compaction of the material had been undertaken, it has been estimated that a direct increase in cost of about 4 to 6 cts per yd would have resulted, causing an increase in the cost of the structure of as much as 70% in some cases. Such an increase has not appeared to be justified for structures designed by empirical methods. Then, too, no selectivity of material has been exercised in the past, and, as the compacting of materials depends to some extent upon their uniformity, efforts toward control of condition and state would not have been marked with great success. Large allowances have been made for the shrinkage of the structures, as follows:

Equipment for construction:	Percentage, shrinkage allowance
Cableway (locally termed "tower machine")	25
Dragline	25
Tractor (wagons, tractrucks, or trucks)	15
Hydraulic fill	8

Uncertainty as to the condition and state of compaction of the material, and assumptions as to conditions and placement that need not be considered generally in earth dam construction, make the design of these structures difficult. For instance, the cohesive strength, as used for the design of side slopes, is usually neglected completely when it is known that draglines or cableways are to be used for the construction of a unit.

The control of seepage through the levee sections has been the prominent feature of their design. This has not been due to a demand for the conservation of water, as is generally the case for dams, but because of the manner in which they have been constructed. The application of the criteria in current use has resulted in designs which have proved satisfactory from the standpoint of stability. It has been shown that most of the material for their construction is silt or a similar soil which is considered as fairly impervious; also that the duration of the flood periods are relatively short as compared to the periods of high reservoir stages for dams. Consequently, the opinion that full seepage

through the sections may not develop seems justified. Through the application of soil mechanics it is possible to attack the problem of design more directly; that is, to determine the stability of the structure on the basis of the strength of the materials used and, in addition, to estimate the seepage and its effect on the stability of the slopes.

The design of a levee, whether based upon empirical or stress-*versus*-strength analysis, depends upon the nature of both the material to be used for its construction and the foundation medium. Consequently, the first step is to explore the materials in the borrow-pit areas by means of borings, so that definite information concerning the available material for construction will be at hand. The material to form the foundation for the structure must be explored in a similar manner. The current practice is to make a system of preliminary borings along lines parallel to the route of the proposed levee, so that the lines divide the area into three parts. The borings are made in pairs on these lines, 1 000 ft apart. The depth of these borings is not great, but it is sufficient to explore the volume of material required for the structure adjacent to their location. In addition, borings are made at 1 000-ft intervals along the center line of the proposed levee, directly opposite those in the borrow-pits, to explore the foundation medium for sufficient depth to assure the non-existence of unconsolidated and apparent unstable material. When such a condition is indicated by the preliminary borings, or the features of the terrain, additional borings are made to trace adequately the extent of such weak formation. The spacing may be reduced to intervals of 100 ft.

The borings for the exploration of the borrow-pit areas and foundations are generally 3 or 4 in. in diameter. They are made with earth augers operated by hand. It has been found that this means of exploration is practicable, permitting the use of unskilled labor and requiring very simple equipment. The samples of the material are obtained directly from the augers as the work progresses and preserved in pint fruit jars which are properly labeled for identification in the classification laboratory.

For the situations requiring special consideration or where no experience has been had with the materials, a primary system of borings is made to supplant the preliminary system just described. The purpose is to provide samples for analysis of the strength of the materials in the borrow-pits and foundations. These samples are obtained from 6-in. borings, using special equipment designed for obtaining undisturbed samples. These large borings are also made, in most cases, by hand-operated augers. A casing of suitable size is used in this work to prevent the caving of the materials penetrated and to exclude the ground-water; thus the materials are sampled "in the dry". The device used for obtaining the undisturbed samples was developed under the auspices of the former Committee on Earths and Foundations of the Society by Professor Gilboy and the writer. The details of this device are shown by Fig. 6. Complete descriptions of the technique of boring and sampling, for both the general and undisturbed samples, are contained in other publications[9] of the writer.

[9] *Proceedings*, International Conference on Soil Mechanics and Foundation Eng., Paper A-23, by S. J. Buchanan, Vol. 2, p. 72–81, June, 1936; "Technique of Soil Testing", by S. J. Buchanan, *Civil Engineering*, Vol. 7, No. 8, August, 1937.

The samples of material obtained by the preliminary exploration are subjected to classification tests, both as to kind of material and general condition. An experienced technician, trained in this work, first classifies the material by visual inspection. The samples are then grouped and such mechanical analyses and water content determinations as are necessary are made of representative members of these groups. Whenever a question arises as to the correctness of visual classification, additional tests are made to dispel any uncertainty. The Atterberg limits, liquid and plastic, are made of a few selected samples so that an index of their general condition, *in situ*, may be had. Thus, the general natures of the materials available for construction, as well as of those in the foundation, are established.

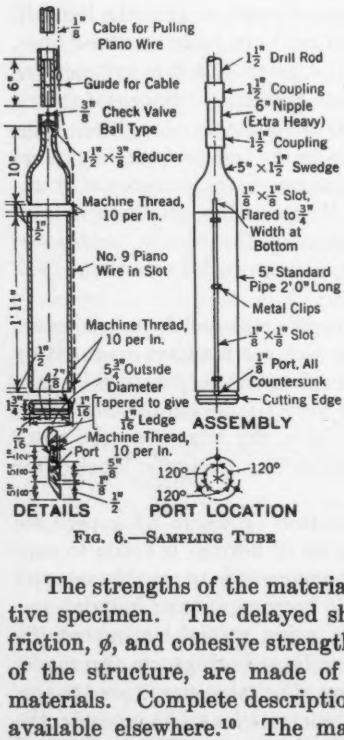

Fig. 6.—Sampling Tube

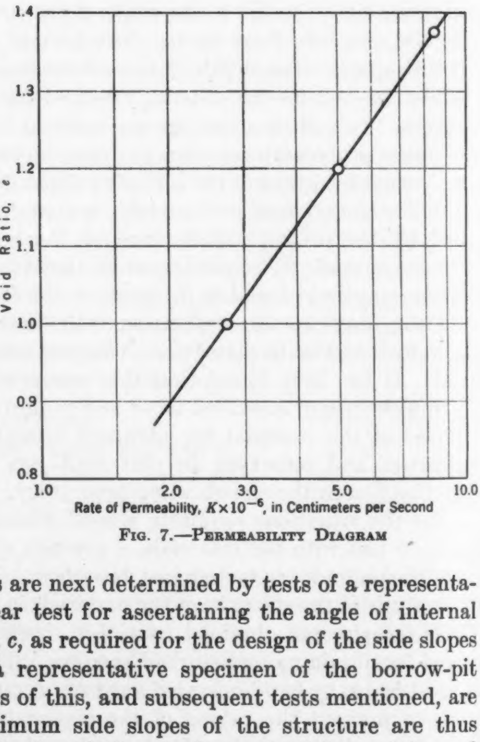

Fig. 7.—Permeability Diagram

The strengths of the materials are next determined by tests of a representative specimen. The delayed shear test for ascertaining the angle of internal friction, ϕ, and cohesive strength, c, as required for the design of the side slopes of the structure, are made of a representative specimen of the borrow-pit materials. Complete descriptions of this, and subsequent tests mentioned, are available elsewhere.[10] The maximum side slopes of the structure are thus established. The strength of the foundation is determined by quick, and squeeze, shear tests of undisturbed samples of the cohesive materials in this medium. Then, the combined consolidation-permeability test is made so that the settlement and under-seepage can be estimated. The final shape of the structure is established, on the basis of information obtained from the latter group of tests.

[10] "Laboratory Procedure in Testing Soils and Sediments", *Bulletin* U. S. Waterways Experiment Station, November 1, 1936.

Basic Data.—The physical features of the problem to be considered are:

Elevation of natural ground surface at the site, in feet (mean Gulf level)	347
Elevation of net grade or crown, in feet (mean Gulf level)	377
Net height of structure, in feet	30
Free-board for maximum designed flood, in feet	1.0
Percentage allowance for shrinkage (construction by tractors)	15
Gross height, in feet, considering shrinkage (30×1.15)	34.5
Thickness of plastic zone in foundation, in feet	23
Factor of safety desired for side slopes	1.5
Factor of safety desired for foundation	1.25

The borrow-pit material is a clayey silt, and other characteristics are as follows:

Percentage of water content (natural state on dry weight basis)	31.5
Void-ratio (natural state on dry weight basis)	1.20
Absolute specific gravity	2.68
Unit weight, in pounds per cubic foot, when placed (judged by experience)	100
Angle of internal friction, ϕ (test result), in degrees	19.4
Cohesive strength, c, in tons per square foot	0.13
Permeability, in centimeters per second (result of tests)	5×10^{-6}

It is evident from the inspection of the conditions indicated by the log of boring, Fig. 8, that the only questionable material, in so far as strength is

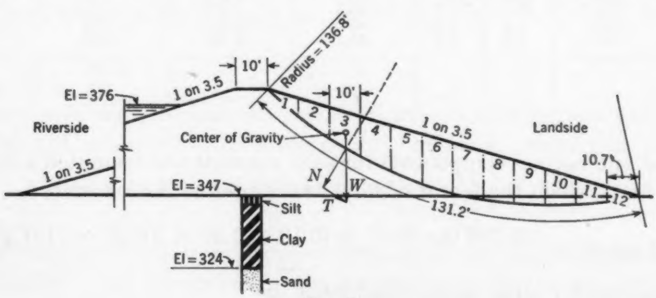

Fig. 8

concerned, is the clay between Elevations 344 and 321. The data obtained from this material are shown as follows: The strength of this material, *in situ* (quick shear tests), equals 0.155 ton per sq ft. Its consolidation and permeability characteristics are as shown in Fig. 9.

Design of Side Slopes.—In the computations for a slope of 1 on 3.5, the position of the center of rotation and the radius of critical arc are both determined by methods referred to previously. These data are shown by Fig. 8. The weight of the material in the structure is assumed to be 100 lb per cu ft. It is assumed that 20% of the cohesive strength of material will be active. The

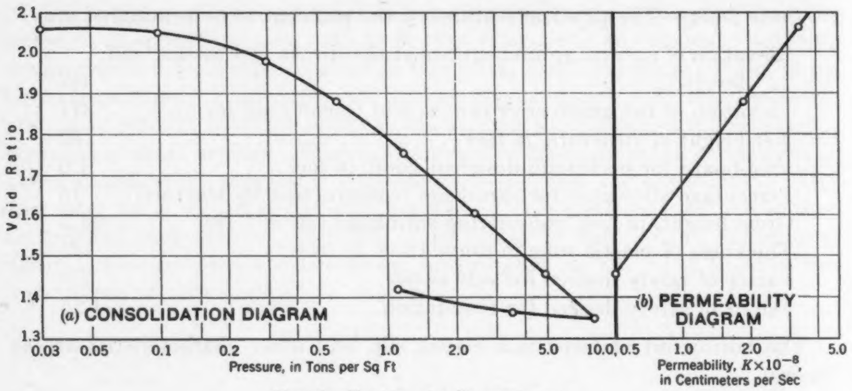

FIG. 9.—FOUNDATION INVESTIGATION

TABLE 3.—COMPUTATION OF TANGENTIAL AND NORMAL COMPONENTS

Segment No. (see Fig. 8)	Width, in feet	Height, in Feet		Area, in square feet	Gravitational force, in pounds	Tangential component, in pounds	Normal component, in pounds
		Left	Right				
1	10.0	0.0	5.7	28.5	2 850	1 980	2 350
2	10.0	5.7	9.8	77.5	7 750	4 320	6 350
3	10.0	9.8	12.8	113.0	11 300	5 640	9 820
4	10.0	12.8	14.6	137.0	13 700	5 750	12 400
5	10.0	14.6	15.6	151.0	15 100	5 460	14 120
6	10.0	15.6	15.8	157.0	15 700	4 600	15 200
7	10.0	15.8	15.0	154.0	15 400	3 230	15 000
8	10.0	15.0	13.6	143.0	14 300	2 050	14 290
9	10.0	13.6	11.3	124.5	12 450	785	12 430
10	10.0	11.3	8.4	98.5	9 850	−80	9 850
11	10.0	8.4	4.8	66.0	6 600	−550	6 550
12	10.7	4.8	0.0	25.7	2 570	−315	2 430
$\Sigma\Delta T$	....	..	..	....	....	32 870	....
$\Sigma\Delta N$	....	..	..	....	....	....	120 790

tangential and normal components for each segment are computed as shown in Table 3, from which the factor of safety is computed, as follows:

$$\text{Factor of safety} = \frac{120\,790 \tan 19.4° + (0.13 \times 0.20 \times 2\,000) \times (131.2)}{32\,870} = 1.51$$

The factors for other slopes considered, are:

Slope	Factor of safety
1 : 3	1.32
1 : 3.5	1.51
1 : 4	1.70

Foundations.—Probable settlement is estimated in the following steps:

(1) The increase in vertical pressure on the foundation, in tons per square foot $\left(\frac{34.5 \times 100}{2\,000}\right) =$ 1.725

(2) The pressure, in tons per square foot, by which the clay stratum has been previously consolidated (estimated from Fig. 9(*a*)) . 0.315
(3) Void-ratio corresponding to initial conditions.................. 2.04
(4) Void-ratio corresponding to final condition.................... 1.64
(5) Thickness of compressible stratum, in feet.................... 23
(6) Total settlement, in feet[11] $= h \frac{e_i - e_f}{1 + e_i} = \frac{23\,(2.04 - 1.64)}{1 + 2.04}$...... 3.0
(7) New gross height, in feet, considering allowance for settlement (= 34.5 + 3.0) 37.5

A recheck of the settlement, using the new gross height (37.5) gives a new value of 3.1 ft, which is sufficiently correct for the purpose; and a recheck of the side slopes shows the factors of safety for the new condition practically unchanged.

The stability of the foundation is determined as follows:

(*a*) Vertical pressure, P, in tons per square foot, created at center of structure $\left(= \frac{37.5 \times 100}{2\,000} \right)$............................ 1.875
(*b*) Thickness, d_a, in feet, of plastic zone in foundation (see Fig. 10).. 23.0
(*c*) Measured strength, in tons per square foot of foundation (test result) 0.155
(*d*) Allowable working strength of foundation material, in tons per square foot, to give a factor of safety of 1.25 $\left(= \frac{0.155}{1.25} \right)$ .. 0.124
(*e*) Base width of structure required, b_b, in feet, $\left(= \frac{P\,da}{\text{Allowable stress}} = \frac{1.875 \times 23}{0.124} \right)$.................. 348
(*f*) Ratio of horizontal to vertical side slope required, allowing for crown width of 10 ft and height of 37.5 ft $\left(= \frac{348 - 10}{2 \times 37.5} \right)$...... 1 : 4.5

Side slopes of 1 : 4.5 are necessary because of the foundation condition, superseding the 1 : 3.5 as required for the material placed in the structure proper. Had the standard side slope for the *B* section, river-side slope 1 : 3.5 (see Table 1), been used, the factor of safety of the foundation would have been 0.98.

Seepage.—The factors involved in determining the height of emergence of the topmost flow line above the base of the levee (using basic equations advanced by Professor Gilboy) are: H = head of water to be impounded; α = angular inclination of land-side slope; b_s = horizontal distance from land-side toe of section to vertical projection of intersection of river-side slope with water surface at a height, H, above the base of the structure; and $m\,H$ = the vertical distance above the foundation of the outlet point of the topmost flow line.

[11] Progress Rept. of Special Committee on Earths and Foundations, *Proceedings*, Am. Soc. C. E., May, 1933, p. 788.

These factors are connected by the following implicit function (see Fig. 10 for values):

$$\frac{2b_s}{H} = m \cot \alpha + \frac{\sqrt{1 - m^2 \sin^2 \alpha}}{m \sin \alpha} - m \sin \alpha \log_e \frac{1 + \sqrt{1 - m^2 \sin^2 \alpha}}{m\,(1 + \cos \alpha)} \ldots (2)$$

The solution of Equation (2) for the value of m, using known values for the factors, b, H, and α, establishes a value of 0.34. The height of emergence above the base is equal to $m\,H = 0.34 \times 29.0 = 9.9$ ft.

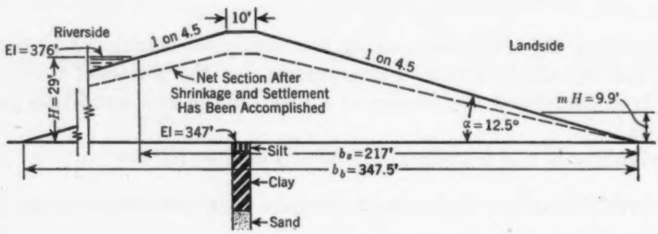

FIG. 10

The quantity of seepage per foot length of levee per second is expressed by:

$$Q = K\,m\,H \sin \alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (3)$$

Substituting the proper values in Equation (3): $Q = \frac{5 \times 10^{-6}}{2.54 \times 12} \times 9.9 \times 0.217$ $= 0.35 \times 10^{-6}$ cu ft per sec per ft of levee, or 0.11 cu ft per min per mile of levee, indicating that little concern is warranted regarding the quantity of seepage to be encountered.

CONCLUSIONS

The present levee system for the Lower Mississippi Valley was designed and constructed at a time when the science of soil mechanics was "in its swaddling clothes". The system as a whole has proved satisfactory under the test of actual use. However, the empirical methods of design were based upon long experience with such structures, the heights of which have increased as have the consequent stresses developed. For subsequent improvements, such as further enlargements or changes in locations that may have become necessary, economical designs may be predicated upon a strength-*versus*-stress basis, to better advantage. Time and funds are no longer necessary to develop empirical designs. The fund of knowledge available permits the problem to be attacked by this new method which has already "won its fighting pants" in other fields.

The net effect of applying knowledge gained through soil mechanics appears to be toward the elimination of such practices as actually producing failures of foundations and later repairing them. Also, it is indicated that the size of the sections may be reduced safely through steepening of the present standard land-side slopes. For new units involving materials with which there has been no experience, savings in time and cost are possible through the application of soil mechanics.

ACKNOWLEDGMENTS

In addition to the acknowledgments and references embodied in the text, the writer wishes to thank the Director of the U. S. Waterways Experiment Station, Paul W. Thompson, Lieutenant, Corps of Engineers, U. S. Army, Jun. Am. Soc. C. E., for his co-operation in making this paper possible. Much valuable assistance was given by the staffs of the Mississippi River Commission and the U. S. Engineer District Office, of Vicksburg, Miss., for which the writer is grateful. He wishes also to acknowledge the assistance of members of his laboratory staff, in particular the following: William L. Wells, Jun. Am. Soc. C. E., Robert M. German, Junior Engineer, and William J. Rowland, Chief Technician, in the preparation of the paper.

QUABBIN DIKE BUILT BY HYDRAULIC-FILL METHODS

By Stanley M. Dore,[12] M. Am. Soc. C. E.

Synopsis

The thorough investigations of materials available for earth dam construction furnish not only the proper information by which to decide the type, dimensions, and cost of the earth dam to be built, but they furnish sufficient data to enable contractors to bid intelligently. Such bidding tends toward more economical and satisfactory construction.

The proper supervision of the construction of a full hydraulic-fill dam is of primary importance. The engineer strives to have the structure built safely and economically, at the same time securing the best quality, the materials available and the difficulty of securing and placing them being carefully considered. The control of construction operations by the engineer to obtain satisfactory results is influenced to a large extent by laboratory investigations and tests of samples from the borrow-pits and from the shoulders and the core of the dam.

The design of a hydraulic-fill dam and the supervision of construction, are influenced considerably by the rate of consolidation of the core material that will occur during and after the sluicing operations. As far as is known there is no existing method of computing this rate of consolidation during construction, other than the semi-empirical method presented herein which was developed from data accumulated on the Quabbin Dike. This method may be used for estimating core consolidation on another project when certain laboratory test data pertaining to that project are available.

Introduction

The Quabbin Reservoir[13] of the Boston, Mass., Metropolitan Water District (capacity, 415 000 000 000 gal), will be formed by a main dam and a dike. Both are full hydraulic earth dams, the former being 2 640 ft long, 170 ft high above the old river bed, and containing more than 4 000 000 cu yd of embankment above the original surface, and the latter being 2 140 ft long, 135 ft high, and containing 2 500 000 cu yd. A typical cross-section of the Quabbin Dike is shown in Fig. 11.

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[12] Associate Civ. Engr., Met. Dist. Water Supply Comm. of Massachusetts, Boston, Mass.

[13] See "Boston's New Water Supply," by Frank E. Winsor, M. Am. Soc. C. E., *Civil Engineering*, June, 1934, p. 283; "Boston Metropolitan Water Supply Extension," by Karl R. Kennison, M. Am. Soc. C. E., *Journal*, New England Water Works Assoc., Vol. XLVIII, No. 2; "Surface and Sub-Surface Investigations, Quabbin Dams and Aqueduct," *Proceedings*, Am. Soc. C. E., March, 1936, p. 297; and, *Engineering News-Record*, June 18, 1936.

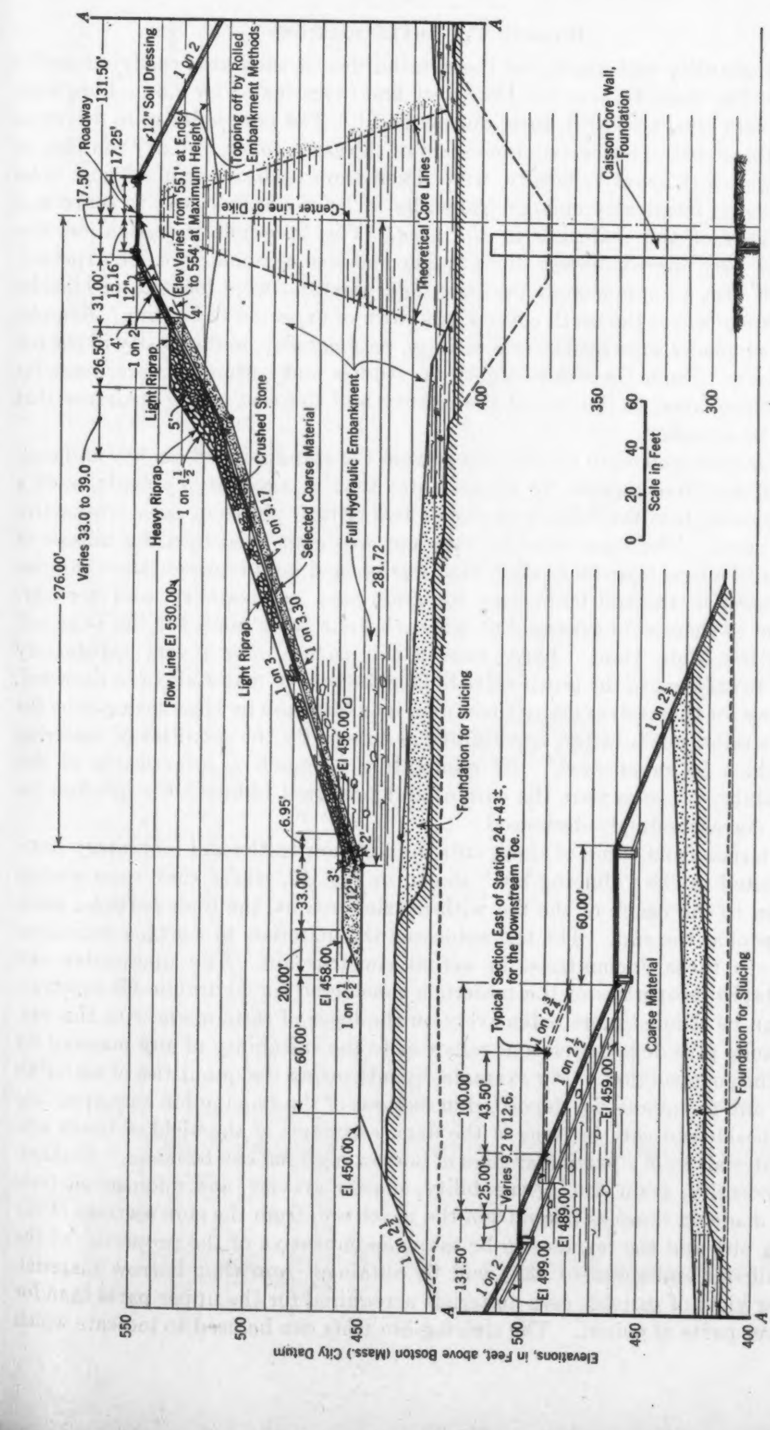

FIG. 11.—TYPICAL CROSS-SECTION OF DIKE EMBANKMENT, QUABBIN RESERVOIR

Borrow-Pit Investigations

The quantity and quality of the over-burden in each area easily accessible to either the Main Dam or the Dike were first investigated by 2.5-in. bore holes and by test pits, 6 to 10 ft deep, dug by hand. The bore holes were driven to reasonable depths to ascertain the extent of the earth cover, and dry samples, at least every 5 ft apart vertically, were taken from each of them. These holes were located frequently enough (generally at least 400 ft apart) to serve as a good index of the materials in the area. The bore-hole investigation also revealed very approximately, the number and size of boulders to be expected. The test pits, to supplement the bore-hole program, were located on hillsides and in areas where the earth cover was known or expected to be thin. Samples from both places were tested for grain size, permeability, and porosity in the soil laboratory. From these investigations, studies and estimates were made for each borrow area, to determine the quality and the cost of the structure that should be obtained.

Areas that seemed to be promising, from the standpoint of quality and cost, were further investigated by shovel cuts; that is, excavations made, with a power shovel, into the hillside at one or two critical places in each prospective borrow area. The excavation of each cut was continued into the hillside or slope until a face from 20 to 30 ft high was opened, and representative samples were tested in the soil laboratory for grain size, permeability, and porosity. The cost of these cuts averaged $1 200 each, four being made for the Dike and ten for the Main Dam. Large cuts of this nature offer a very satisfactory exhibit to indicate to the prospective bidder the type of materials to be expected. They pay for themselves many times over on a large job by eliminating from the bidder's mind much of the uncertainty as to exactly the qualities of materials with which he must work. By eliminating as much as practicable of this uncertainty, the sum that the careful and preferred bidder adds to offset his risk, is correspondingly decreased.

Materials from some of these cuts, in addition to the soil laboratory tests, were tested in the "sluicing bin" shown in Fig. 12, where they were washed down on to the beach of the bin with nozzle streams, the finer particles going into a pool at the end. The test subjected the materials to working conditions similar to those encountered in actual construction. The qualitative and quantitative properties of the materials considered for hydraulic-fill construction, can be studied more extensively on the basis of tests made with this bin. The results give definite relative values as to the suitability of any material for use in the construction. For example, by measuring the quantities of materials sluiced and the quantities deposited in sections of the sluicing bin corresponding to the beach and core sections of the dam, estimates of the yield of beach and core materials from a particular class of borrow material can be made. Furthermore, porosity, grain size, permeability, specific gravity, and microscopic tests can be made on samples taken from the beach and from the core sections of the sluicing bin, and the results can be taken as indicative of the properties of the hydraulic-fill embankment that will be obtained from that borrow material. A larger yield of suitable core materials is required for the upper parts than for the lower parts of a dam. The sluicing-bin tests can be used to indicate which

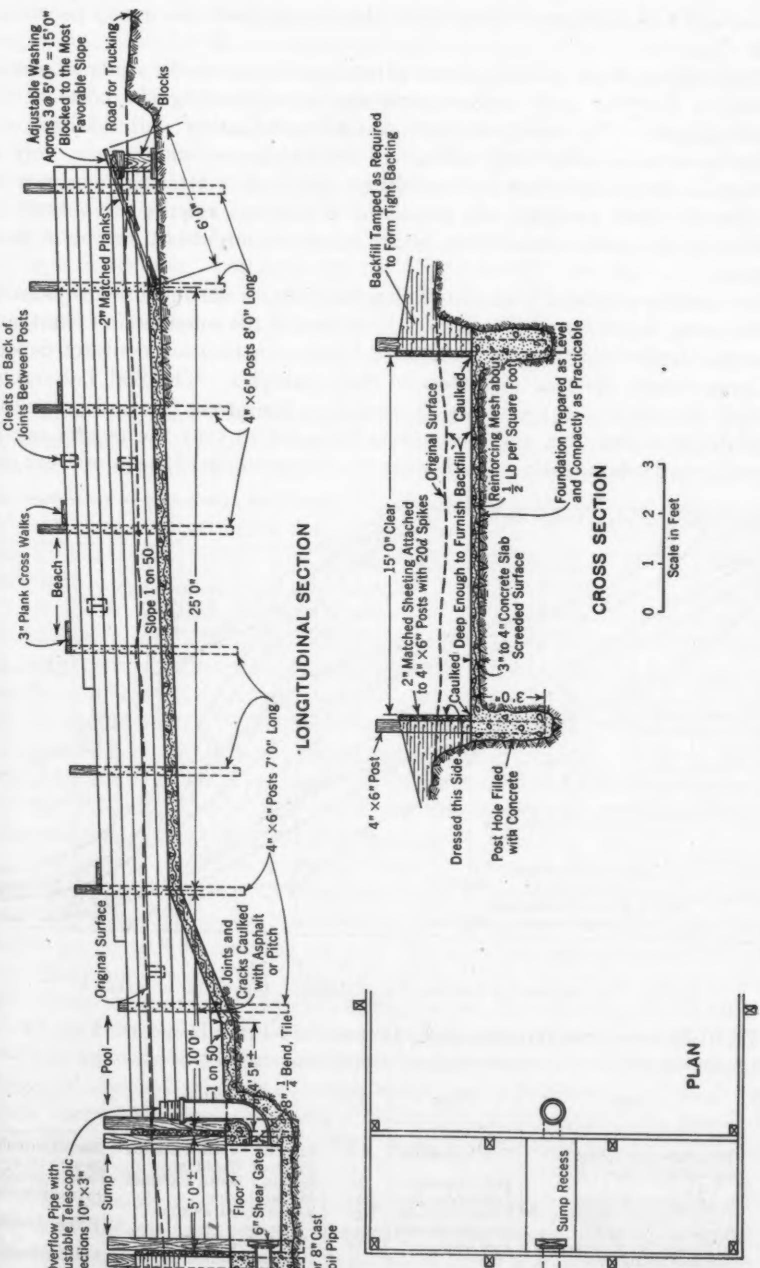

FIG. 12.—SLUICING BIN TEST

material or which mixture of materials is the proper one to use in any particular height.

The results of these investigations aided appreciably in the study of several methods of building each embankment and in estimating the costs of the various layouts. The methods that seemed satisfactory and feasible, and practicable economically, were offered to the bidders as suggestions, only to aid them in tendering a fair and intelligent proposal. After the contract for each dam had been awarded, the successful bidder was required to submit for approval his own plan of operation, which might, or might not, be one of those suggested.

The samples obtained from bore holes, test pits, or shovel cuts are valuable in influencing the design of the dam or the layout of the construction plant, and the proper design and the prices obtained in the construction contract depend to a large extent upon an adequacy of these samples. The Soil Laboratory enhances the value of the samples by analyzing the properties and presenting the results in a classified and summarized manner so that the results can be used easily and intelligently. Such work eliminates much of the guesswork and

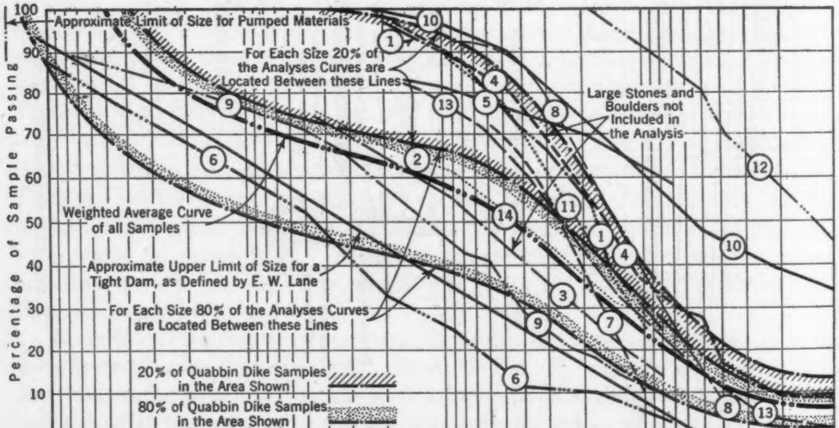

FIG. 13.—COMPARISON OF BORROW-PIT MATERIALS, IDENTIFIED IN TABLE 4

TABLE 4.—IDENTIFICATION OF HYDRAULIC-FILL DAMS IN FIG. 13

Curve No.*	Dam	Type	Curve No.*	Dam	Type
1	Henshaw, California	Full hydraulic	8	Linville, North Carolina	Semi-hydraulic
2	Swinging Bridge,† New York	Full hydraulic	9	Tieton,† Washington	Semi-hydraulic
3	Dwinnell, California	Full hydraulic	10	Saluda,† South Carolina	Semi-hydraulic‡
4	Davis Bridge, Vermont	Semi-hydraulic	11	Magic, Idaho	Semi-hydraulic
5	Somerset, Vermont	Semi-hydraulic	12	Alexander, Hawaiian Islands	Full hydraulic§
6	Conconnully, Washington	Full hydraulic	13	Cobble Mountain, Massachusetts	Full hydraulic
7	Paddy Creek,† North Carolina	Semi-hydraulic	14	Quabbin Dike, Massachusetts	Full hydraulic

* See Fig. 13. † Average of tests. ‡ Minor slip. § Serious slip during construction.

difference of opinion that accompanies the visual examination. If soil laboratory results and the classification of samples for various dams already built are available, comparative studies can be made of the materials available for construction and of those used elsewhere.

Fig. 13, with Table 4, shows a comparison of grain-size analyses curves for borrow-pit materials used in Quabbin Dike with those for some other large dams. A "full hydraulic" dam is an earth dam built by transporting the material on to the dam by pipes or flumes, in suspension in water and depositing it in the dam by water, a separation of sizes taking place during deposition. A "semi-hydraulic" dam is similar to "full hydraulic" except that the material is transported on to the dam in some other manner, usually in a "dry" state in cars, trucks, or wagons.

Control During Construction

To control the deposition of material in a hydraulic-fill dam is a complex process, and many features must be properly co-ordinated to obtain the best results. The analysis of "hog-box" samples and of core and beach samples at regular intervals is a valuable part of this control. Periodic tests for grain size, permeability, and porosity give indications that the structure is, or is not, being built satisfactorily, and they reveal the properties of all parts of the structure.

By the analysis of the hog-box samples the builder is enabled to anticipate the borrowing operations, and the engineer is enabled to regulate them properly so that poor construction conditions and results may be avoided. These analyses indicate, directly, to both builder and engineer, any lack of "coarses" for the shoulders or of "fines" for the core, any presence of too large a quantity of any one size, and other unsatisfactory size conditions. Any conditions of size deficiency or excess can be remedied or corrected as soon as it is indicated by the laboratory analyses, by borrowing materials of differently tested characteristics from other parts of the borrow area.

The analyses of core samples, taken at frequent intervals from the dam itself, indicate definitely the existence of any coarse tongue or intrusion into the core or fine deposit in the shoulders so that corrective measures may be taken before construction advances too far. By co-ordinating the result of rod-pipe soundings made along the edges of the core and pool with the size analyses made from the materials there, an inspector can learn to judge the quality of the core material by the resistance offered to the penetration of his rod. Thus, he can feel any coarse tongue that projects into the core. With routine testing the grain-size test acts as a constant check on his judgment. If these tests are made reasonably close (say, every 100 ft along the edges of the pool) and are supplemented by the rod-sounding work, unsatisfactory conditions can be ascertained for correction.

Glacial sands and gravels usually have such high internal frictional resistances, that shear tests are not necessary to the design, because a conservative cross-section obtained by using reasonable up-stream and down-stream slopes normally has ample volume and weight to withhold the pressures from the core during and, of course, after construction. For some types of materials, such as

those obtained from alluvial deposits, shear tests should be used extensively to determine the qualities of the shoulders of the dam or the foundations under it, and in such cases the results will control many features of the design. However, an occasional shear test for any type of material is a valuable check of the qualities already judged suitable by visual inspection. In the case of control during construction, shear tests on the core material of a hydraulic-fill dam have a definite utility, as the frictional value of the core increases with the consolidation. If, from tests, the shear is determined for various degrees of consolidation of the core material, the safe rate of progress of construction and the rate at which the width of the core can be decreased safely can be controlled more intelligently. At Quabbin Dike the shearing resistance of the core materials, even a few days after being deposited, was sufficiently large so that it was unnecessary to limit the rate of construction or the rate of decreasing the core width.

Core samples should be analyzed for moisture contents (or porosity) at frequent intervals to determine the rate at which the core consolidates during construction. These specimens can be obtained by sinking a pipe well into the core and taking samples, at various elevations, in as near an undisturbed condition as practicable. Although the core materials may vary in properties over a wide range, the results of the moisture content determinations, in conjunction with grain-size analyses curves, will show a definite trend of the rate of consolidation.

At Quabbin Dike a steel pipe of 0.5-in. plate, 36 in. in diameter, was built vertically in the core during the construction (see Fig. 14) and this well was equipped with ports through which samples can be taken at various elevations. If desired the samples can be taken several feet away from the observation well by the use of long tubes. Samples from the ports of this well were taken at regular intervals (approximately one month) and tested for grain size and porosity. The results indicate that certain definite laws operate to control the rate at which the core materials consolidate during construction.

In the very first stages of the consolidation considerable water is released from the part of the core near the bottom of the pool through vertical springs into the pool; but for parts of the core covered substantially, the drainage is mostly horizontal. The edges of the core consolidate more rapidly than the center for two reasons: First, they are nearer the coarse, and free-draining, parts of the dam, and the water squeezed out of the pores is quickly carried away; and second, they usually contain the coarser of the core deposits which will yield its water relatively quickly because the permeability coefficients are correspondingly higher.

Disregarding this more rapid consolidation of the edges, and assuming that there is only one consolidation rate for the core over its entire width, for each increment of height, the degree of consolidation during construction can be expressed by an equation involving the quantity of water squeezed from the core material, to effect a certain consolidation, as follows:

$$Q = K \frac{H}{0.25\,b} A\,t \qquad (4)$$

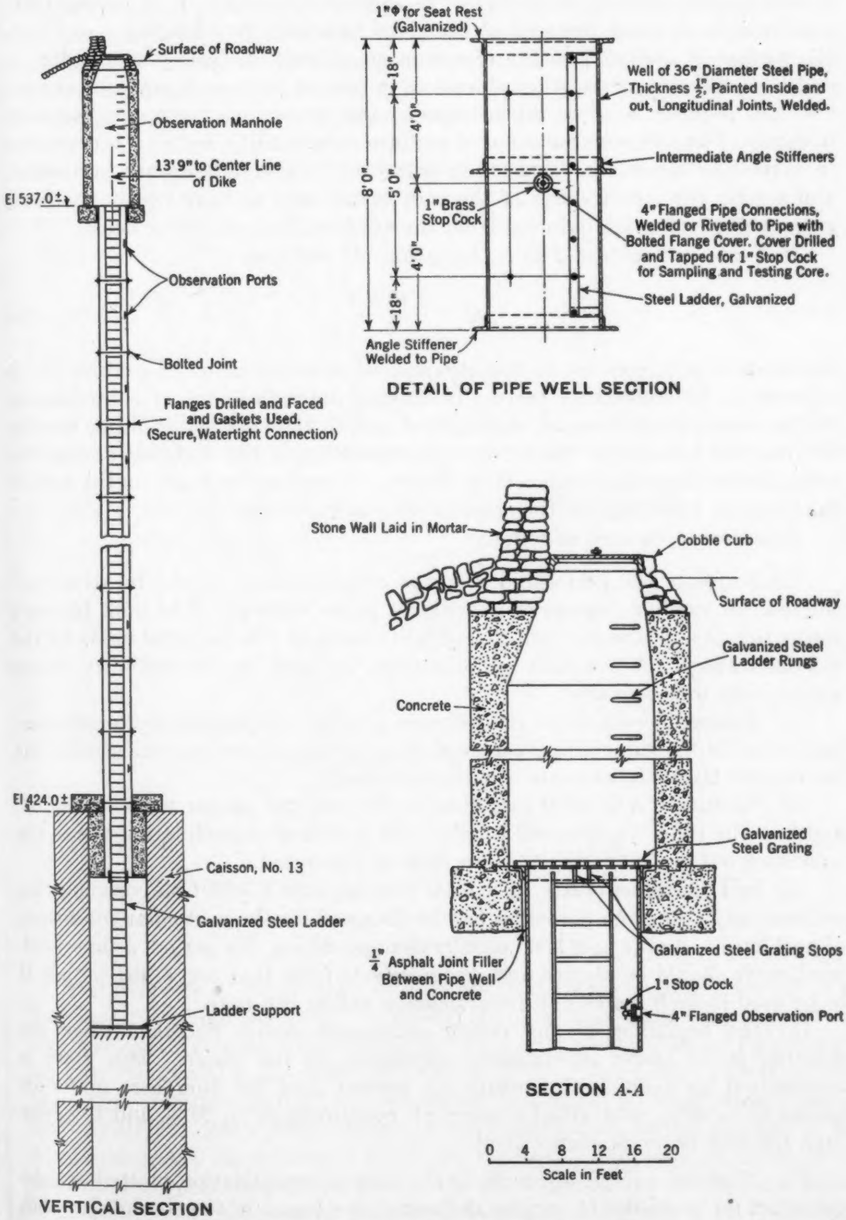

FIG. 14.—DETAILS AND SECTIONS OF OBSERVATION WELL

in which Q = the rate of discharge, in cubic feet per day; K = permeability coefficient of the core material at the given location; H = head of water from the surface of the pool to the elevation considered, in feet; b = breadth, or width, of the core at the given elevation, in feet; A = area, in square feet; and t = time required to drain the necessary water to secure a given consolidation, in days. The core material is not of uniform permeability, but as it is deposited in horizontal layers, the changes in permeable characteristics are horizontal, and a strip the entire width of the core would tend to have nearly the same permeability although different from the adjacent strip above or below.

If A = a unit area of 1 sq ft, Equation (4) becomes,

$$Q = \frac{4\,K\,H\,t}{b} \qquad (5)$$

Although it is known to be a mathematical approximation, Equation (5) is believed to be sufficiently correct in making determinations of consolidation during construction because, during that period, the core material has usually not reached a state in which the compressibility of the material affects the consolidation rate to an appreciable degree. Observations made on the core of the Quabbin Dike indicate the practicability of its use (see Fig. 15).

Equation (5) is used as follows:

(1) Estimate the permeability of the core material, for the location considered, for varying degrees of porosity or water content. The data for such estimation are obtainable from permeability tests on core material made by the soil laboratory. A constant value cannot be used, as permeability varies appreciably with porosity.

(2) Assume increments of reduction of porosity as consolidation progresses, and solve for the value of ΔQ corresponding to such reductions (the smaller the increments the more accurate the determination).

(3) Substitute ΔQ for Q in Equation (5) and the proper values of K, H, and b. The time, Δt, required to effect the increment consolidation due to the squeezing out of the water, ΔQ, can then be computed.

(4) Add the values of Δt. The total time required to effect the consolidation between any two given porosities can be obtained for the particular conditions of pool level. As the pool level usually changes during this period, a time-pool-level curve should be plotted and, progressively from that curve, the proper H to be used in each increment determination can be estimated.

(5) The beginning of any period considered should be taken when the material is no longer in complete suspension in the water. This state is represented by a different porosity for coarser than for finer core materials (about 65%, 60%, and 55% for materials containing 30%, 20%, and 10% less than the 0.01 mm size, respectively).

Fig. 15 shows a series of curves of the time of consolidation plotted against porosities for materials of various characteristics found in the core of Quabbin Dike. The curves were computed theoretically by means of Equation (5). The circles indicate observed porosities of samples taken from the core, and the line and arrows indicate the difference between the theoretical and the observed.

The differences between the two are minor in the upper parts of the diagram, and the use of Equation (5) should be limited to construction conditions in which the core materials are in a highly saturated state, and in which the degree of consolidation has not progressed far enough to permit great support of the overlying load to be carried by the solids of the core.

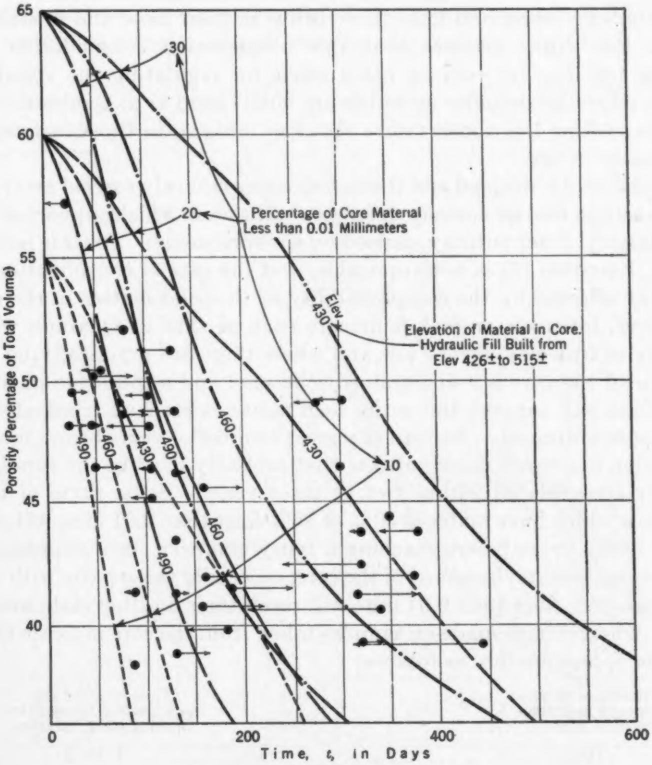

Fig. 15.—Consolidation Diagram

The load above any volume of core materials is transmitted through it in two ways: Through the water, which is compressed and squeezed out as fast as the permeability of the core will permit; and, after sufficient relief by drainage of such excess water, through the solid particles of the core. The transition from the condition of full load carried by the water to load carried almost entirely by the core solids is usually an extremely gradual one because of the slow drainage of the tight core materials. However, the average construction condition is such that the entire load is not transmitted through the water. This state is one that exists when the solids are suspended in the water and occurs only in the materials in or near the bottom of the pool. This suspended state is quickly relieved by vertical drainage into the pool itself. The core materials are then in a state such that the solids are not suspended, but rest

lightly upon each other, receiving some light and partial support through the solids beneath. It is this condition, or a similar one, which exists in the core as long as the pool is maintained full of water for the subsequent construction operations above; and Equation (5) is usable for just such conditions. This is the reason why H is the head of water. The excess weight of the liquid core over that of the water is neglected because of the partial support the solids obtain from each other and from those below in their loose and unstable state.

Thus, the writer believes that this construction consolidation formula (Equation (5)) can be used on other dams for regulating the speed of construction, when the shoulder materials are not as good as at Quabbin; or it may be used in making less conservative shoulder designs, in those instances, than are at present in use.

After the pool is drained and the excess water is slowly carried away, there is a slow transition into an entirely different condition in which support of the load above is largely, if not entirely, carried by the core solids. For this latter state, of course, Equation (5) is not applicable, and the rate of consolidation then is appreciably affected by the compressibility of the solid matter in the core.

However, for a silt or rock-flour core such as that at Quabbin where the percentage of true clay is very low and where there are practically no colloids, the texture of the core is a fine-grained solid one; and a condition in which the core particles will support the entire load above is reached relatively quickly after the pool is drained. Instead of a great number of years before most of the consolidation is secured, tests indicate that probably most of the core is almost completely consolidated within two or three years. Some parts of the core, such as those which have material 30% or 35% finer than 0.01 mm, will probably settle for many years before reaching a fairly compact state approaching the limiting or stable state; but most of the core, especially those parts with material coarser than 20% finer than 0.01 mm, will reach their limiting state much more quickly. Observations made on samples taken from the core indicate that such periods are approximately as follows:

Percentage of core material finer than 0.01 mm	Time required for consolidation to reach its limiting state, in years
10	1 to 2
15	1.5 to 3
20	3 to 5

In each case t is measured from the time the pool is first drained of water. Comparisons made with the consolidation formula[14] introduced by Glennon Gilboy, Assoc. M. Am. Soc. C. E., indicate that much longer periods would be expected but the result of tests of the core materials of Quabbin Dike leads the writer to believe that the rate of consolidation of solid-grained core materials of specific gravity of 2.6 to 2.7 similar to those in Quabbin Dike will be much more rapid than Professor Gilboy's formula indicates.

The consistency of core materials in a hydraulic fill when a fairly stable state of consolidation is reached, has been the subject of much discussion. Observations made to date (1937) on the core indicate that, for material of a

[14] "Mechanics of Hydraulic Fill Dams," by Glennon Gilboy, Assoc. M. Am. Soc. C. E., *Journal*, Boston Soc. of Civ. Engrs., July, 1934, p. 187.

size 25% finer than 0.01 mm, or coarser, the stable state will be at least that of a stiff putty. Even the finer portions will be in a putty-like state, but softer; and there will be no such condition as a permanent liquid or semi-liquid core condition.

The impression that the lower parts of the core consolidate more quickly, due to the pressures from above, is not entirely true, because the width of the core is narrower near the top and consolidation occurs quickly at low pressures because the excess water can soon reach the permeable shoulders. Observa-

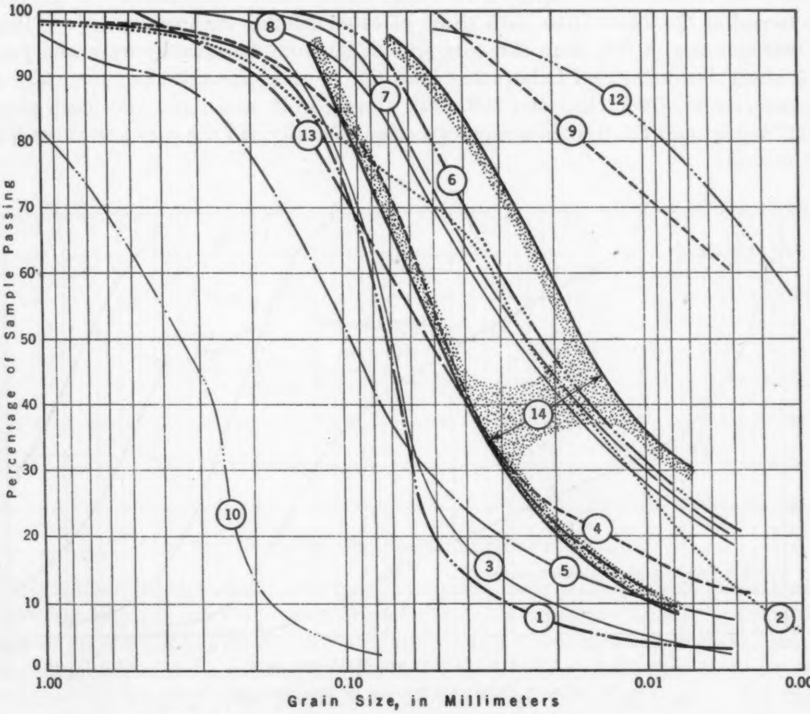

FIG. 16.—COMPARISON OF CORE MATERIALS, IDENTIFIED BY TABLE 5

TABLE 5.—IDENTIFICATION OF HYDRAULIC-FILL DAMS IN FIG. 16

Curve No.*	Dam	Type	Curve No.*	Dam	Type
1	Henshaw, California	Full hydraulic	8	Huffman, Ohio	Full hydraulic
2	Swinging Bridge, New York	Full hydraulic	9	Calaveras, California	Full hydraulic and semi-hydraulic
3	Dwinnell, California	Full hydraulic			
4	Davis Bridge, Vermont	Semi-hydraulic	10	Soft Maple, New York	Full hydraulic
5	Bridgewater, North Carolina	....	12	Alexander, Hawaiian Islands	Full hydraulic
6	Germantown, Ohio	Full hydraulic	13	Cobble Mountain, Massachusetts	Full hydraulic
7	Lockington, Ohio	Full hydraulic	14	Quabbin Dike, Massachusetts	Full hydraulic

* See Fig. 16.

tions to determine the porosities of core samples indicate that after the pool has been drained, the rate of consolidation is fairly uniform throughout the core and that the pressure due to the depth does not then cause faster consolidation in the lower sections probably because, with the greater width, a relatively longer time is required to carry away the excess water even if it is under a greater pressure.

The qualities of the hydraulic fill obtained from the borrow-pit materials indicated in Fig. 13 are summarized in diagrammatic form in Figs. 16 (with Table 5), 17, and 18. Fig. 16 with Table 5 shows also a comparison of the core material of Quabbin Dike with those of other similar structures. More than 8 000 samples (4 500 from the core and 3 500 from the beach) were analyzed for grain size in the Soil Laboratory and Fig. 19 gives the average percentage of these samples finer than the 100-mesh sieve (0.149 mm), the 200-mesh sieve (0.074 mm) and the 0.01-mm size for various locations in the core and shoulders of the dike.

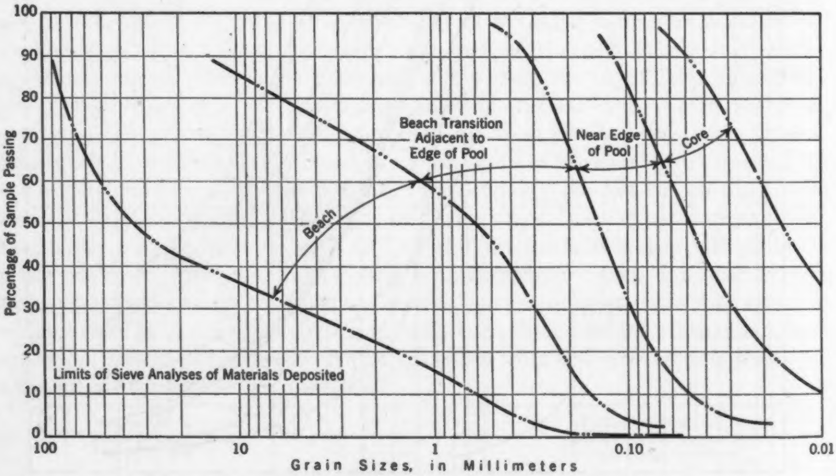

FIG. 17.—GRAIN SIZES; MATERIALS IN BEACHES AND CORE

SEEPAGE ESTIMATES

The quantities of water that will seep through the earth core of Quabbin Dike will be extremely small, but estimates were made to determine reasonable requirements for water-tightness of the core materials and reasonable dimensions to which to build the core, the qualities of the available borrow materials considered. The flow net method[15] was used in these estimates. It consists of establishing the location of the seepage flow lines, from them determining the equipotential loss-of-head lines, and computing the seepage from the latter in terms of the assumed permeabilities of the core and beach materials. The result is expressed in terms of K, the coefficient of permeability of the core materials; and by substituting the values of K that are considered as reasonable

[15] See 1er Congrès des Grand Barrages-Scandinavie, Vol. III and IV, Juin-Juillet, 1933.

in the result, seepages for different qualities of cores can be secured. The locations of the flow lines were made mathematically by "cut and try" methods, and were checked experimentally by use of a laboratory model similar in some general respects to that described in 1934 by Hibbert M. Hill, Assoc. M. Am. Soc. C. E.[16]

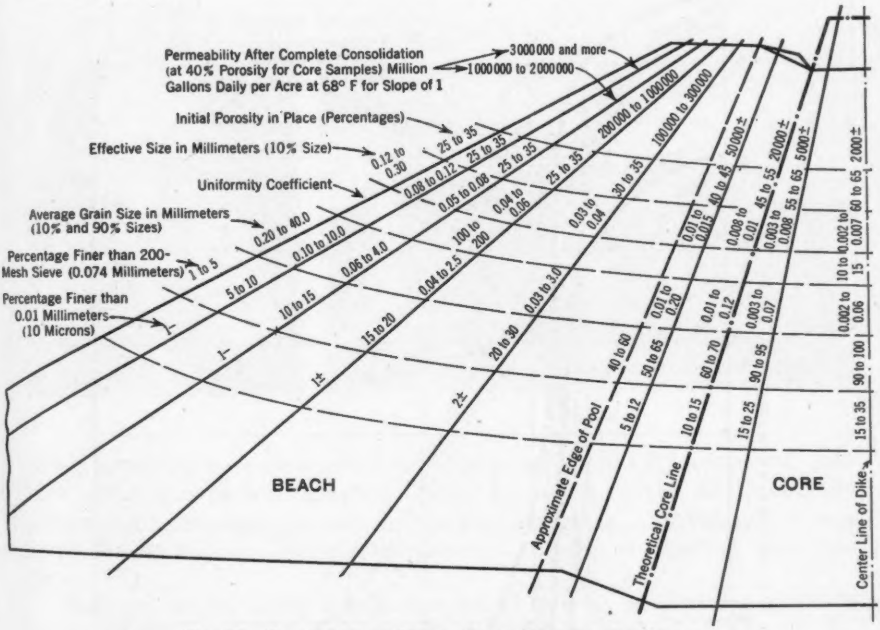

FIG. 18.—PROPERTIES OF MATERIALS IN BEACHES AND CORE

This model was a small water-tight wooden tank with a heavy plate-glass front (see Figs. 20 and 21). As the beach materials are many times as permeable as the core, the slope of the flow lines up stream and down stream from the core are exceedingly flat; and a satisfactory check on the location of the flow lines can be obtained by finding their position in and through the core itself. For this reason the sections tested were those of the core only. Several sections of core were considered, and their patterns were cut from aluminum plates about 0.1 in. thick.

When tested, the plate was set next to the glass front, and held at a uniformly short distance from it by metal shims. It was then backed up by a wooden, water-tight bulkhead which divided the tank into two parts. Water to the equivalent reservoir level was maintained in one-half the tank and to the equivalent tail-water level in the other half.

The first tests showed that laminar or stream-line flow was obtained throughout when the spacing between the glass front and the core plate was 0.006 in., or less, although a spacing of 0.012 in. could be used with only a small amount of turbulent flow occurring near the caisson cut-off. For greater

[16] *Civil Engineering*, January, 1934, p. 32.

spacing the area of turbulence increased in size. As the seepage of water through soil is laminar or stream-line flow, the spacing during testing was limited to less than 0.006 in.

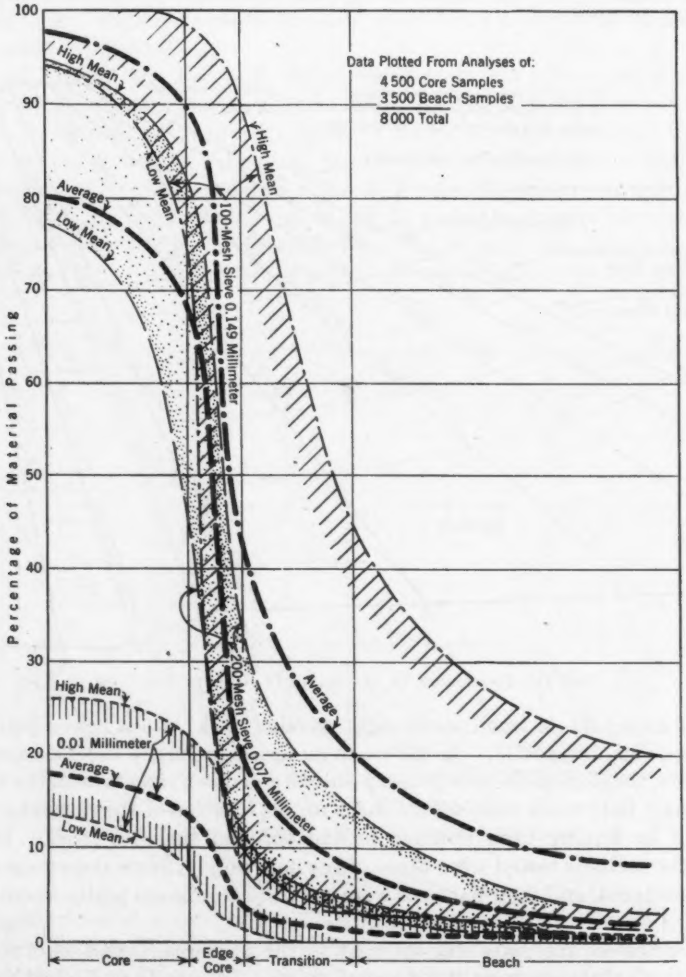

FIG. 19.—SIEVE AND HYDROMETER ANALYSES RESULTS

Some difficulty was experienced with air bubbles and with capillary action near the top of the core section. The former was controlled, to some extent, by running the water until the temperature of the tank was the same as that of the water. The latter was eliminated by use of a metal shim, shaped experimentally to reduce the effect of capillarity on the flow lines without depressing or otherwise unduly affecting their location. This shim was placed between the plates at the top of the core near the water level and prevented any water from being

raised into the space between the plate and the glass above the upper normal laminar flow line, which was determined experimentally.

As Figs. 20 and 21 indicate, a dye was introduced into the water through small openings just up stream from the core plate. This dye would travel

Fig. 20

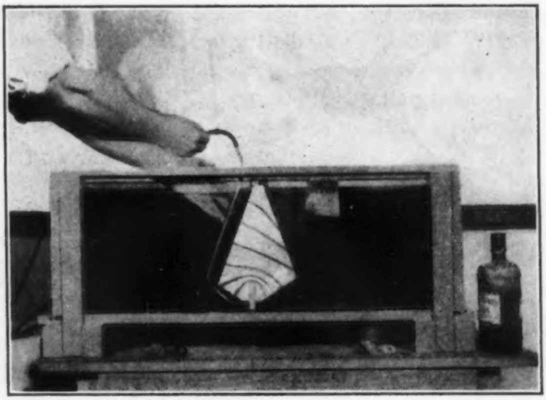

Fig. 21

slowly across the core plate along the seepage lines, and the locations of these lines could thus be photographed. They checked with the mathematically determined locations in an extremely accurate manner. Turbulent flow was easily ascertained by observing the action of the dye in traveling across the plate.

The core section finally chosen was 100 ft wide at an elevation of 120 ft below the full reservoir level, with side slopes of approximately 3 on 1. Increasing this width of 100 ft to 120 ft would have reduced the seepage only about 7%, decreasing it to 60 ft would have increased the seepage about 33 per cent. Because of the very small seepages considered, the width chosen was determined principally by the more important factor, the probable minimum yield of suitable core material from the available borrow materials.

Stability During Construction

Estimates of stability of the Quabbin Dike were made for many assumptions of conditions. These estimates led to the development of a nomographic chart (see Fig. 22) for solving in a short time the rather cumbersome Gilboy formula:[14]

$$\sqrt{r_w} = \frac{(C_s - A_c)\sqrt{1 + B^2} + \sqrt{C_s - A_c}\sqrt{C_s - B}\sqrt{1 + A_c^2}}{(1 + C_s^2) - (C_s^2 - A_c)(C_s^2 - B)} \ldots\ldots (6)$$

in which r_w = the ratio of the unit weight of core material to the unit weight of the shoulder material; A_c = co-tangent of the angle of the core slope with the horizontal; B = co-tangent of the angle of internal friction of the shoulder material; and C_s = co-tangent of the angle of the outer slope, with the horizontal.

If the dimensions of the core and beach sections and the unit weight of the core and shoulder materials are known or assumed, the internal friction or shear value necessary for stability can be estimated quickly from the chart; or, if the qualities of the shoulder and core materials are assumed, the outer slopes of the dam required for cores of assumed or required dimensions can be computed.

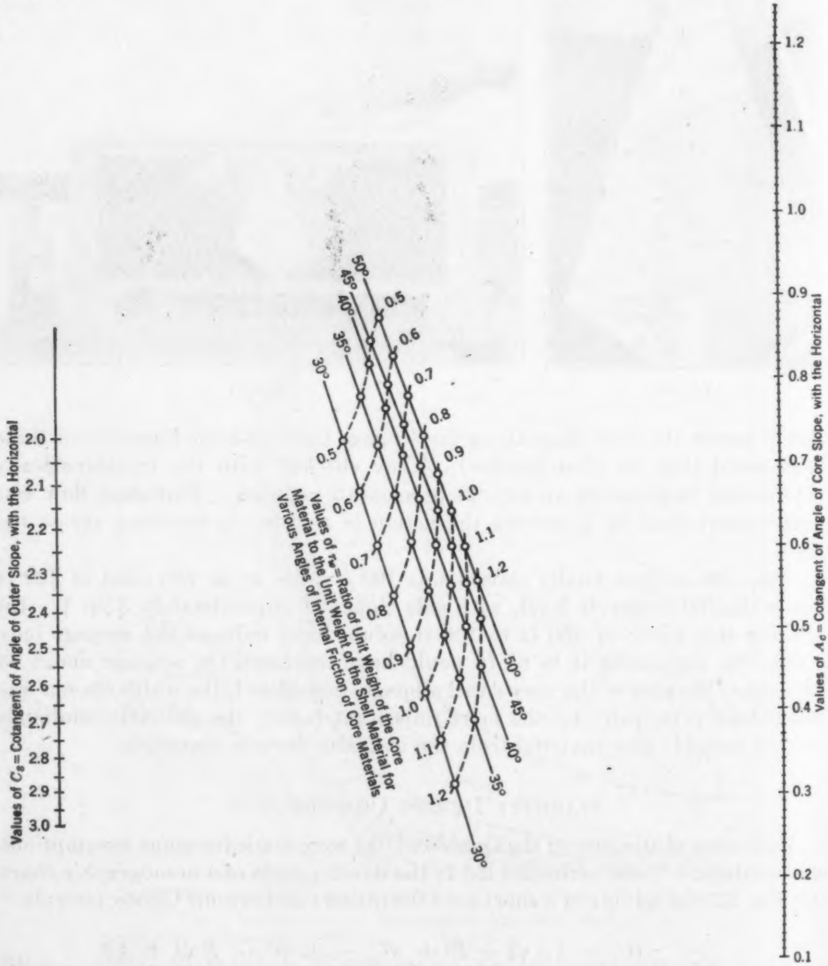

FIG. 22.—STABILITY NOMOGRAPH FOR HYDRAULIC FILLS

Equation (6) is based on the assumption that the core is liquid throughout, which is considerably on the safe side; but if the estimator has data upon which to base an assumption that only some fraction of the full liquid pressure is being developed for the conditions in mind, Fig. 22 can still be used by multiplying the ratio, r_w, directly by the percentage of the full liquid pressure that is existent, and using the corrected value of r_w in the graphical solution.

It will be noted that A_c and C_s are the usual slope constants, representing the ratio of horizontal run to vertical rise. Thus, an outer slope of 1 on 2.5 corresponds to a value of C_s equal to 2.5. Values of C_s are indicated on the scale on the left side of Fig. 22; values of A_c are on the scale of the right side.

The procedure for using Fig. 22 is as follows: Assume that the width of core at Elevation 410 was 100 ft and that the hydraulic-filling has progressed to the point at which the core has reached Elevation 450 with a width of 69 ft. This corresponds to a core slope of 1 on 0.387, which, if continued upward, would intersect the center line at Elevation 540.5, from which point the outer slopes are closely approximated by a line having a slope of 1 on 2.76, as shown in Fig. 23.

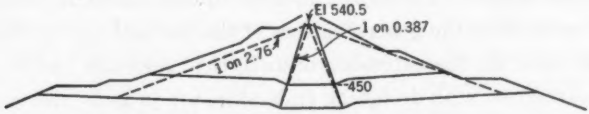

FIG. 23

Assume that the ratio of unit weight of core to unit weight of shell is 1.1 and that the angle of internal friction of the shoulder material is 45 degrees. It is desired to determine whether the section is stable.

A straight-edge is placed on 2.76 on the scale at the left and on 1.1 on the 45° line in the center of Fig. 22. This intersects the right-hand scale at 0.7 and indicates that for the conditions assumed the section is stable, since the values of A_c in the right-hand scale are maximum values for stability. Conversely, the values for C_s at the left are minimum values.

The beach materials at Quabbin Dike in the outer portions of the shoulders weigh as much as 135 lb per cu ft and average probably about 120 to 125 lb per cu ft. The core materials weighed about 100 lb per cu ft when first deposited and from 110 to 125 lb per cu ft when the dam was completed. For stability based on the assumption of full liquid pressure from the core, an internal friction angle of less than 30° in the shoulder materials was required. These shoulder materials had a tested friction angle of more than 45 degrees.

ACKNOWLEDGMENTS

The writer wishes to acknowledge the aid of many members of the Engineering Department of the Metropolitan District Water Supply Commission, of which Frank E. Winsor, M. Am. Soc. C. E., is Chief Engineer, and Karl R. Kennison, M. Am. Soc. C. E., is Assistant Chief Engineer, in the preparation of data used in connection with this paper. He is particularly indebted to Clarence R. Day, Assoc. M. Am. Soc. C. E., in connection with work on the seepage model, Coleman C. McCully, Assoc. M. Am. Soc. C. E., in connection with work on the stability diagram, and Jerome L. Spurr, Assoc. M. Am. Soc. C. E., in connection with soil laboratory sampling and testing of materials.

STABILITY OF EMBANKMENT FOUNDATIONS

By B. K. Hough, Jr.,[17] Jun. Am. Soc. C. E.

Synopsis

The analyses made in connection with the design of two large rock-fill embankments proposed as part of the now discontinued Passamaquoddy Tidal Power Project, included certain unusual studies of stress distribution in clay foundations and the character of embankment settlement resulting from foundation over-stress. The report of these studies, which is presented herein, is intended to describe the general nature of the findings and to draw attention to a possible need for the extension of analytical methods (which, at present, are chiefly concerned with designing embankments so as to prevent over-stress of the clay type of foundations) to new methods in which a condition of over-stress is accepted for economic reasons, and the extent of the resulting embankment settlement is computed as the basis for cost estimates.

Introduction

In the field of Soil Mechanics, which itself is a new and specialized branch of Civil Engineering, a certain degree of internal specialization already seems to be in progress. One of the most recent manifestations of this process is the rapid growth of a group the members of which are chiefly concerned with the problems involved in the construction of large earth or rock-fill embankments.

One general and obvious requirement confronting an engineer in designing an embankment is that, after completion, the proposed embankment shall be stable. This involves, first, a determination of the physical characteristics (notably shearing strength) of the foundation and embankment material; then, a calculation of the stresses created within the embankment and extending from the embankment down into the foundations; and, finally, a judicious comparison of strength with stress in the light of past experience and a full realization of the limitations of the analytical methods and the test data which he had used.

Both experience and theory now seem to indicate that analyses for stability vary widely in character and complexity with the type of material encountered in the underground, and the nature (that is, both the material and the design) of the embankment. The soil over-burden at a dam site is generally either predominantly cohesive or cohesionless, and most embankments, with particular reference to their so-called impervious cores, may be classified in the same manner. The four possible combinations of conditions resulting from such classification represent four fairly distinct types of problems, each of which requires a certain degree of special treatment. Additional complications are introduced by the various methods of building embankments, such as rolled or hydraulic fill, and if rolled, whether compacted to a high or low relative density. The scope of this paper is limited to a discussion of certain studies

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which have relation to a definitely cohesionless uncompacted embankment founded on a relatively uniform, cohesive foundation. Methods of laboratory testing, model studies, and analysis of data performed in the U. S. Engineers Soils Laboratory for the Passamaquoddy Project are described, since it is believed that certain experiments performed there may point to a new conception of the action of embankment foundations. The work done at Eastport, Me., on this problem was predominantly of a qualitative nature, and it is hoped that the findings as reported herein will stimulate further and more detailed studies.

Foundation Exploration

In leading up to the subject of this paper, the following description of the drilling operations in the foundations is given as a matter of general interest. In Eastport, the problem of foundation investigation for the two largest dams presented unusual difficulty. Known, respectively, as the Eastport and Lubec Dams, these structures were intended to close two channels or passages between the ocean and the waters of Cobscook Bay, the high level pool of the proposed power development. The unusual difficulty lay in the fact that each of these channels was approximately 3 500 ft wide, that in each channel the tidal range had a magnitude of about 26 ft, and that the resulting flow of water occurred with a velocity sometimes as great as 6 ft per sec. The soil over-burden to be explored lay at the bottom of these channels under depths of water as great as 100 ft, and from a somewhat limited investigation previously conducted, was believed to be predominantly a deposit of soft marine clay. In the exploration described herein, one drill hole was actually driven to Elevation −274.5 before rock was encountered.

Drilling for exploratory purposes, and to obtain undisturbed soil samples suitable in size and character for compression and shear tests, was conducted from two large derrick-boats, one operating in each channel. To protect the 6-in. drill casing from the force of the tidal currents and to provide a stable support for the drill platform, an extra heavy 21.5-in. casing or spud was first erected in sections over the sides of these boats. All guys to the top of the spud and all anchor lines to the boats were led to hand winches, constantly attended so that the spud could be kept vertical and the position of the boat maintained at all times, despite the rise and fall of the floating plant with the tide. The bottom of the spud was allowed to penetrate the over-burden sufficiently to give it lateral stability, while the weight of the spud and drilling equipment was carried by a large, circular plate clamped to the spud a short distance above its lower end so that it had bearing against the surface of the over-burden. After erection of the spud, the drill rig was mounted on the platform provided at the top and drilling was conducted in normal fashion in a 6-in. casing inside. A general view of the barge in actual operation in the Eastport Channel is shown in Fig. 24. In this work the importance of obtaining undisturbed soil samples for accurate foundation studies was realized and close inspection of the condition of all samples was made in the laboratory. Some comment is made subsequently on the effect of disturbance of samples, as indicated by the test data obtained in the Eastport Laboratory.

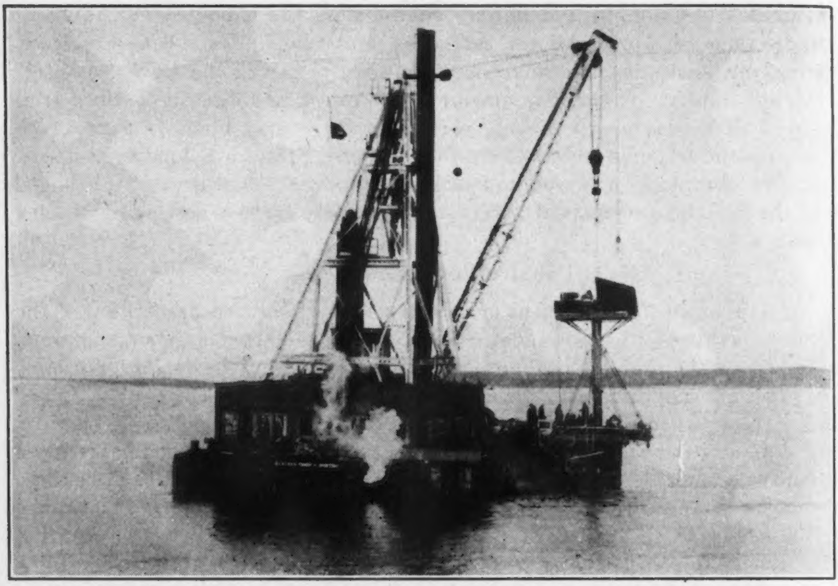


FIG. 24.—DEEP-WATER DRILLING, PASSAMAQUODDY TIDAL POWER PROJECT; 160-FOOT SPEED IN 120 FEET OF WATER

LABORATORY TESTING

Routine soil testing on this project was performed by methods which in general character are similar to those in use in the Soil Mechanics Laboratory of the Massachusetts Institute of Technology, Cambridge, Mass. The equipment used for performing the shear and consolidation tests was the same in principle

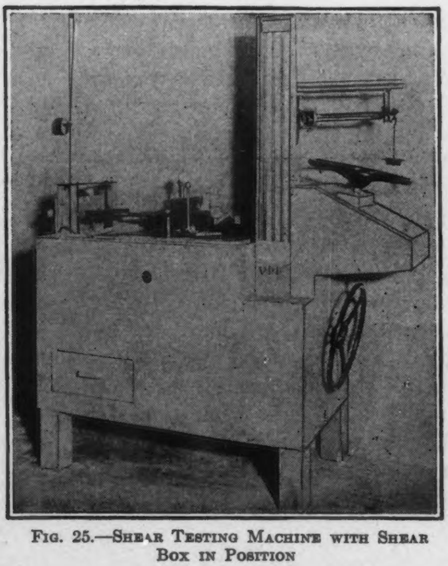


FIG. 25.—SHEAR TESTING MACHINE WITH SHEAR BOX IN POSITION

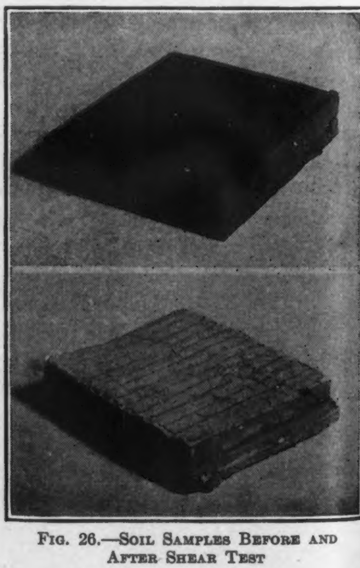


FIG. 26.—SOIL SAMPLES BEFORE AND AFTER SHEAR TEST

as that designed by Glennon Gilboy, Assoc. M. Am. Soc. C. E., for the U. S. Engineer Laboratory, in Zanesville, Ohio.[18] The shear machine, shown in Fig. 25, is operated in such a manner that the rate of horizontal strain of the sample during the test is constant. The particular rate chosen for standard tests on clay samples of the type shown in Fig. 26 was that which was found to give the most conservative value of shearing strength for the type of material tested. A rate of strain commonly used was 0.05 in. per min. In testing cohesive samples, a relatively light vertical load was used, in general equal to about one-fourth the horizontal pull. No effort was made to determine an apparent angle of internal friction for clay samples and no increase in natural shearing strength was allowed as a result of possible consolidation during the construction of the embankments. The shear tests were run simply to determine the natural shearing strength of the deposit of cohesive material at the site of the proposed structures.

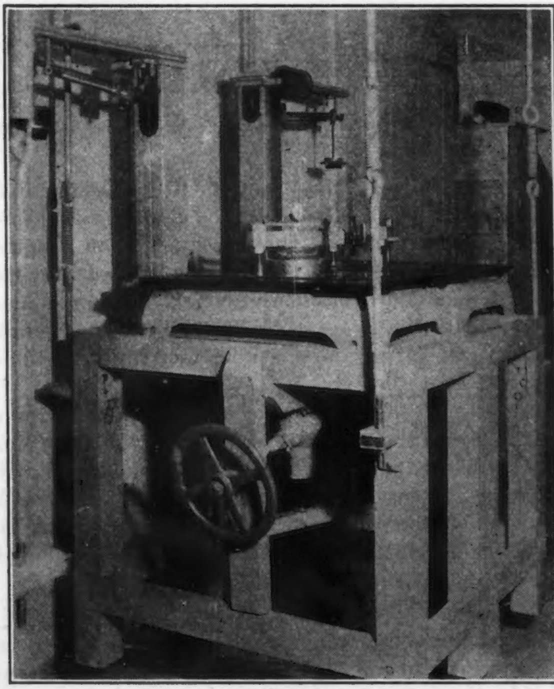


FIG. 27.—VIEW OF CONSOLIDATION MACHINE

At this point it seems advisable to acknowledge the fact that most of the statements made in the foregoing paragraph touch on highly controversial subjects. In making these statements, it is hoped rather than feared, that they will provoke discussion and constructive criticism. Consolidation tests were performed with the equipment shown in Fig. 27, and the method used for

[18] *Engineering News-Record*, May 21, 1936.

plotting test data was that developed[19] by Donald W. Taylor, Assoc. M. Am. Soc. C. E.

Effect of Sample Disturbance

Correlation of the data from shear and consolidation tests was found to give an indication of the degree of disturbance suffered by individual samples during the drilling operations. Fig. 28 shows stress-strain curves, *A* and *B*,

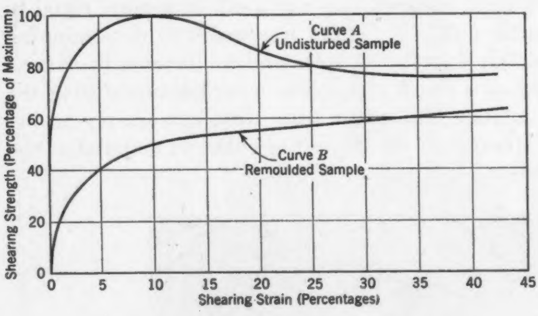


Fig. 28.—Stress Strain Shear Curves

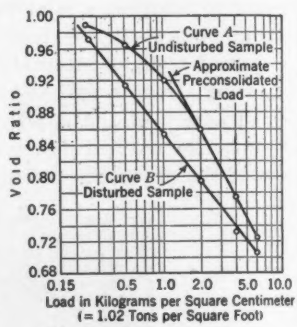


Fig. 29.—Pressure, Void-Ratio Curves

respectively, for an undisturbed sample and for the same sample retested after complete remoulding. It will be noted that the character of the curves is quite different. It was decided, therefore, that when a test on a supposedly undisturbed sample yielded data which when plotted had the appearance of Curve *B*, the sample should be regarded with some suspicion. Curves *A* and *B* in Fig. 29 are pressure, void-ratio curves for consolidation tests on undisturbed and remoulded samples, respectively, and also show noticeably different characteristics.

Attention is invited to the foregoing considerations and the suggestion is made that through a careful examination of test data in some similar manner, a judgment may be reached as to the value of the sampling methods in use on any particular project. More attention in the future than has been given in the past to this detail of laboratory testing is believed to be advisable and may lead eventually to the development of more dependable sampling technique.

Summary of Test Data

Correlation of drilling and testing records made possible the preparation of the profiles shown in Figs. 30 and 31. The information given in these diagrams may be amplified by the statement that, in general, the shearing strength of the clay in the Eastport Channel tends to increase with depth, whereas the reverse condition exists in the Lubec Channel. The average values in both cases range between 0.1 and 0.2 ton per sq ft, although in the Eastport Channel lower values predominate and in the Lubec Channel a few values as high as 1.0 ton per sq ft were found. The average water content for samples from both channels is

[19] "Improving Soil Testing Methods", by Glennon Gilboy, Assoc. M. Am. Soc. C. E., *Engineering News-Record*, May 21, 1936.

about 30% of the weight of solids. The general variation of water content with depth was found to be in agreement with the variation of shearing strength, since in the Eastport Channel it decreases with depth and in the Lubec Channel it increases. A mechanical analysis of the material from both channels showed

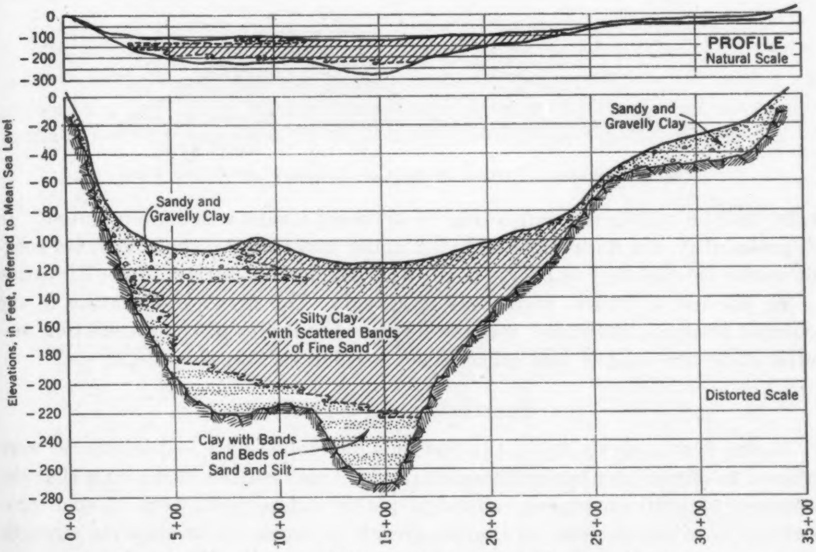


FIG. 30.—PROFILE ALONG CENTER LINE OF DAM, EASTPORT CHANNEL

that it had a colloidal content of about 20% and about equal percentages of clay and silt. Its specific gravity is approximately 2.75.

PROPOSED STRUCTURE

On this material it was proposed to build a structure of the type shown in Fig. 32. It was intended to construct the rock-fill section of these dams first as a separate operation in order to effect closure of the channels and, after this

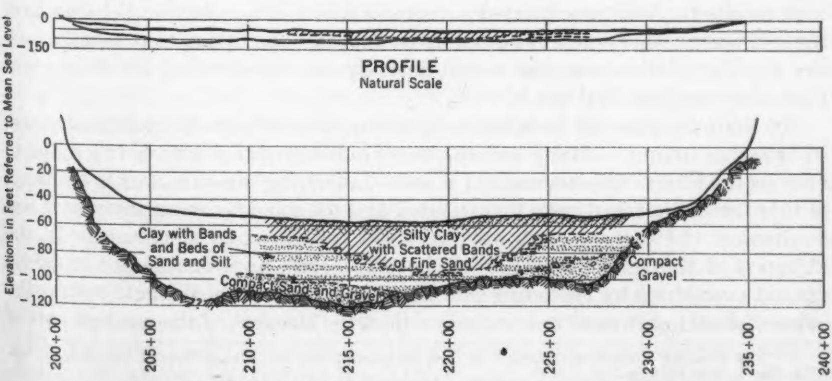


FIG. 31.—PROFILE ALONG CENTER LINE OF DAM, LUBEC CHANNEL

step, to seal the dams with a relatively impervious soil blanket placed on the "up-stream" face. It should now be evident that the previous reference to a definitely cohesionless embankment is justifiable in connection with these structures. Nearly all the material in these dams would be placed in flowing

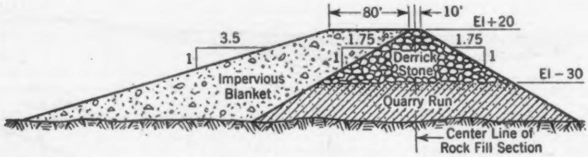


FIG. 32.—ORIGINAL TYPICAL SECTION OF EASTPORT AND LUBEC DAMS

water and no rolling or compacting by artificial means was contemplated. In all probability, the finest sizes included in the lower part of the completed rock-fill would be classified as coarse sand, whereas, in the section above Elevation − 30, the use of 20-ton stone was proposed. The first consideration in the required analysis, therefore, was the degree of stability of the foundation material under the load of this cohesionless rock-fill section alone.

PRELIMINARY ANALYSIS

In the first analysis made, the methods developed by Leo Jurgenson[20] were utilized to obtain an approximate solution. This resulted in showing that the proposed rock-fill structures, although partly submerged, were of such proportions as to create shearing stresses greatly in excess of the shearing strength of the foundation material. Comment on the applicability of Jurgenson's method to this particular problem is deferred to the latter part of this paper. Considerable importance, however, was attached to the preliminary finding that in order to achieve stability of the proposed rock-fill, the base width would have to be increased from five to ten times. The additional quantity of fill material required for this alternate design was found to be so considerable that the possibility presented itself of effecting an economy by building the embankment with the relatively steep slopes originally planned and letting failure and displacement of the underground take place, even if eventual settlement to rock resulted. This precipitated a discussion as to the extent and character of the settlement which might reasonably be expected, assuming that as indicated, the clay foundation material would actually be over-stressed before the embankment reached final grade.

In contemporaneous foundation problems, the writer believes this situation to be rather unique. Many embankments and highway fills have failed due to unexpected lateral displacement of a soft, underlying stratum, but few, if any, of these structures had been investigated and designed by the principles of soil mechanics. In such cases as have received treatment by these methods, the objective of the analytical work has been to attain an actual factor of safety against over-stress by flattening side slopes to a degree which would reduce the induced shearing stresses to a value less than the strength of the weakest part of

[20] "The Shearing Resistance of Soils," by Leo Jurgenson, *Journal*, Boston Soc. of Civ. Engrs., Vol. XXI, No. 3, July 1934, p. 242.

the foundation. A definite lack of precedent was found, therefore, on procedure for an embankment analysis in which failure and at least partial displacement of the underground was actually contemplated, although as an exception to this statement the special case of settling highway fills by dynamiting soft underlying material may be cited.

Model Studies

The available information was so limited as to the type or extent of settlement to be expected, that some experimentation was considered necessary as a preliminary to deciding on a method of attacking the problem. Already set up in the laboratory was a simple polariscope (shown diagrammatically in Fig. 33)

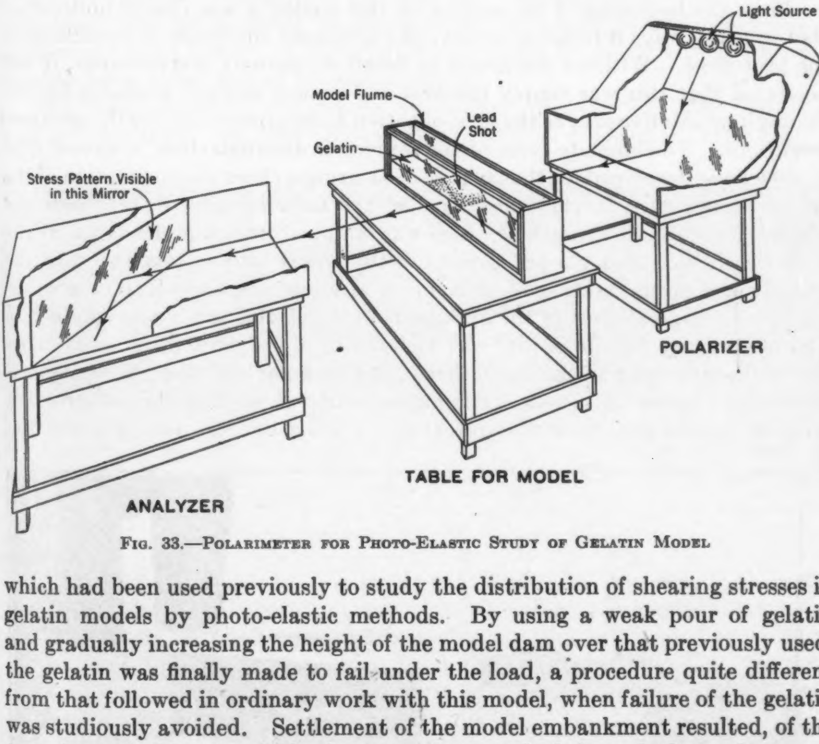


Fig. 33.—Polarimeter for Photo-Elastic Study of Gelatin Model

which had been used previously to study the distribution of shearing stresses in gelatin models by photo-elastic methods. By using a weak pour of gelatin and gradually increasing the height of the model dam over that previously used, the gelatin was finally made to fail under the load, a procedure quite different from that followed in ordinary work with this model, when failure of the gelatin was studiously avoided. Settlement of the model embankment resulted, of the type shown in Fig. 34. This view was photographed through the analyzer and besides showing the shape and outline of the embankment section after failure, it also shows a number of stress bands in the gelatin beyond the toes, where evidently the gelatin was not overstressed. Interesting though this last point may be to some, the particular object of introducing Fig. 34 is to show the character of deformation that occurred under the embankment. It is quite clear that, in this case, settlement was not maximum under the center line, although numerous predictions to this effect had been made, but rather it is apparent that maximum settlement occurred at points on either side of the center line, about half-way out toward the toes. This experiment was repeated

a number of times under a fairly wide range of controlling conditions until it was established as being typical. From this work, the type of settlement obtained was termed "heart shaped", due to its resemblance to the lower part of an inverted heart.

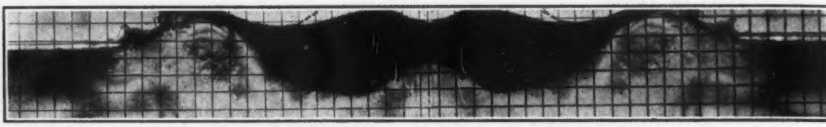


FIG. 34.—SETTLEMENT OF MODEL EMBANKMENT

From the beginning of the work with this model, it was clearly understood that in many ways it failed to achieve the necessary similitude to conditions in the prototype. Without discussing in detail its obvious shortcomings, it will be stated that this was simply the first and easiest method available for obtaining a qualitative idea of the type of action to be expected under the assumed conditions. To eliminate some of the sources of dissatisfaction, a second type of model was developed for the purpose of securing a closer approximation of the actual conditions. In this second model the material used to represent the underground was clay, much of which was obtained from samples taken at the actual dam sites, and the proportions of the model were changed so that the axial length of the section of dam being studied was considerably greater. Fig. 35 is a general view of the apparatus in which this study was conducted. The preliminary test procedure was to build up a bed of uniform, remoulded clay in the rather shallow tank and then, using irregular lead slugs, to construct a model dam across the center, following as nearly as possible the construction program contemplated for the prototype. During the test, movement of the

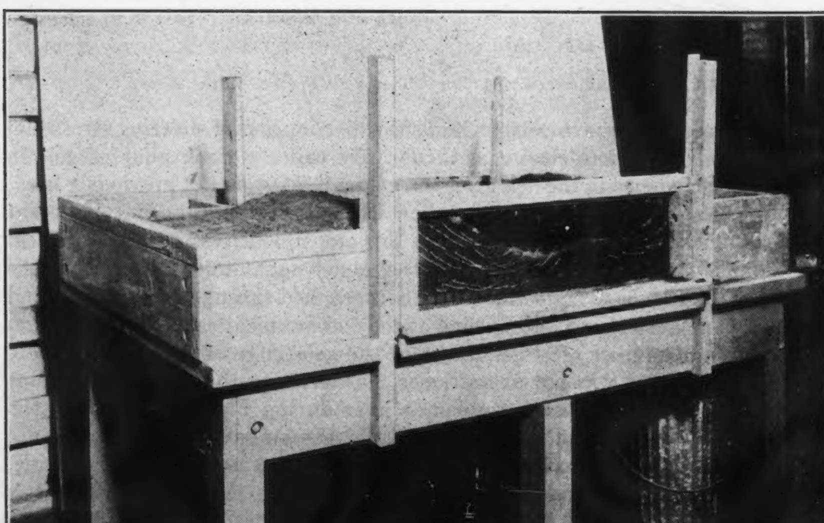


FIG. 35.—TABLE FOR MODEL STUDY OF SETTLEMENT CHARACTERISTICS OF PROPOSED STRUCTURES

clay under the embankment load could be watched through a window in one side of the tank.

To facilitate this observation, small white reference points were inserted between the clay and the window at the intersections of a grid system, prior to loading. Successive positions of each point were marked on the glass so that at the end of each test their individual paths were determined. At the center of the model, in a section perpendicular to the axis of the dam, a series of vertical reference lines was inserted. The deformation of these lines could not be observed during the test, of course, but were exposed afterward by removing the embankment and cutting the model along the mid-section, a view of such a section being shown in Fig. 36. The originally vertical reference lines may be seen clearly, and the profile of the clay is shown to have assumed the previously mentioned "heart shape". Also, plainly visible, are the so-called "mud wave" formations on either side of the embankment.

This confirmation of the results previously obtained, although still of a qualitative nature, was very encouraging. In this connection, attention is

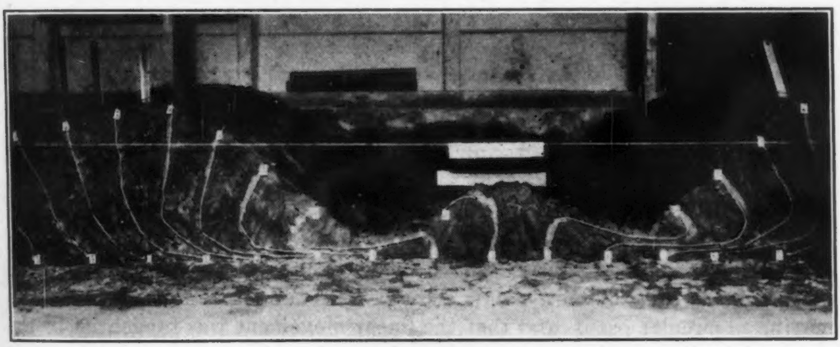


FIG. 36.—CROSS-SECTION OF SETTLEMENT UNDER MODEL DAM

invited to the experience of A. Casagrande, Assoc. M. Am. Soc. C. E.[21] and Professor Gilboy,[22] who noted a similar type of failure under embankments on soft foundation material. Sufficient confidence in the accuracy of the general character of this work was gained from this accumulation of evidence to proceed with the development of a quantitative method of analysis for use in solving the particular problem previously outlined.

HAINES METHOD

Continued experimentation with the clay model and careful observation of its action during test, led to the development of a method of stress analysis before failure, and a method of determining the extent of settlement after failure, of proposed embankments. The details of this analytical method were developed by R. M. Haines, Jun. Am. Soc. C. E.

[21] "The Shearing Resistance of Soils", by Leo Jurgenson, *Journal*, Boston Soc. of Civ. Engrs., Vol. XXI, No. 3, July, 1934; Discussion by A. Casagrande, p. 976.

[22] "Stability of Embankment Foundations", by Glennon Gilboy, Assoc. M. Am. Soc. C. E., *Transactions*, Third World Power Conference, Washington, D. C., 1936.

A characteristic set of records for the clay-model testing is shown in Fig. 37. Inspection of this diagram led to the conclusion that movement of the reference points was largely circular under the dam and became linear under the mud

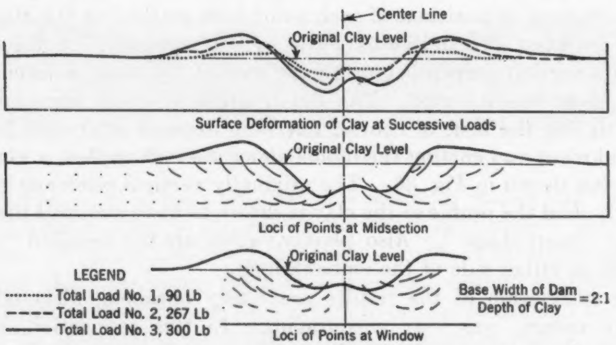
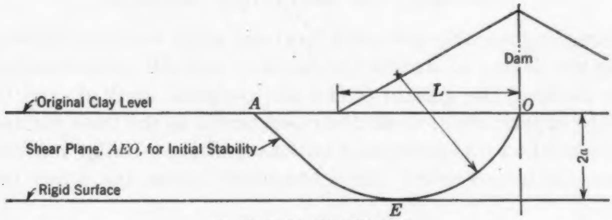


FIG. 37.—MODEL STUDIES OF SETTLEMENT AND SHEAR FAILURE CHARACTERISTICS

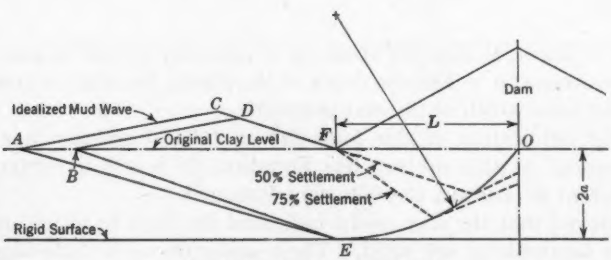
waves. This type of movement was recognized as being similar in general character to that predicted by Professor Dr. H. von Krey, of Berlin, Germany, for the movement of plastic material under long footings. Development of the Haines method proceeded from this observation to the task of making an analysis of the conditions of equilibrium along a failure surface also similar to that defined by Krey, but in this case located by trial and error in a manner somewhat resembling the work of K. E. Petterson, member of the Swedish Geo-Technical Commission in 1914–1922. Location of the failure surface and development of the statical analysis were facilitated by finding that the movement of the reference points indicated a common center for all points moving in circular paths, and that the linear movement was tangential to the circular. Failure surfaces for a ratio of $\frac{L}{a} = 4$ are shown in Fig. 38. It will be noted that that balancing effect of the mud wave is taken into account in Haines' analysis and that one effect of this mud wave is to cause the failure surface to move outward as settlement progresses. Statical analysis of the surface in Fig. 38(*a*) was made to determine whether or not a given section would be initially stable, whereas analysis of surfaces, such as those in Fig. 38(*b*), assuming in succession various settlements, was made to determine how much settlement could be expected when initial stability was not indicated. (In Fig. 38(*b*), Line *BEO* is the shear plane for 50% settlement; Line *AEO* is the shear plane for 75% settlement; Line *BDF* is the mud wave for 50% settlement; and, Line *ACF* is the mud wave for 75% settlement.) The same kind of study was also made for trapezoidal embankment sections in order to make possible analysis of various construction stages.

The types of analysis indicated in Fig. 38 were planned on the assumption that the half section of embankment tended to rotate about the center of the circular sliding surface and thereby caused a rotational movement of the foundation material included within this surface and an upward movement

involving passive pressure of the material along the straight part of the failure surface. By equating the moments of the forces tending to cause motion with those tending to resist it, an expression was obtained relating the intensity of loading at the center line of the embankment with the cohesion or inherent



(a) INITIAL STABILITY



(b) EXTENT OF SETTLEMENT

FIG. 38.—SURFACES INVESTIGATED FOR EXTENT OF SETTLEMENT

shearing strength required for a factor of safety of unity. This expression can be simplified to the form,

$$p = K\, s_c \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which K is a variable with values as shown in Fig. 39.

For the case in which initial stability is not indicated and it is desired to compute the probable amount of settlement of the embankment into the foundation material, the following formula, was derived:

$$p = K\, s_c + K_i\, a\, w \quad \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

Fig. 40 is a set of curves for the solution of Equation (8) for the assumption of 50% and 75% settlement. It will be noted that solutions for various stages of construction are indicated from 25% of the final height of the embankment to the full height or the triangular section.

Solution of settlement problems by the Haines method as indicated by Figs. 39 and 40 involves a process of trial and error. If initial stability is not indicated, a solution is obtained for stability at 50% settlement. If this does not indicate stability, 75% settlement is assumed, and if stability is still not indicated, the assumption is made that the material will settle to rock.

No greater refinement is warranted by the assumptions involved in developing this method. One consideration, for example, is the extent to which

the shearing strength of the foundation decreases as failure occurs. It is possible that additional investigation, however, may eventually modify the approximate nature of this method.

Discussion of Analytical Methods

In the foregoing remarks reference has been made to three different methods of analyzing the degree of stability of earth or rock-fill embankments, namely, Jurgenson's method, the gelatin model-photo-elastic method, and the Haines' method. The experimental work described herein as the basis for development of the Haines method was performed under conditions similar to those assumed by Jurgenson for his so-called "rigid boundary" case, for which he proposed the formula,

$$s = \frac{p\,a}{L} \qquad (9)$$

in which s = principal shearing stress; p = intensity of load at the center line of the embankment; a = half the depth of the plastic foundation material; and, L = half the base width of the embankment.

Since the publication of this formula[20] in 1934, Jurgenson has presented other discussions on this subject, but Equation (9) is still in current use to a sufficient extent to warrant the following comment.

It is believed that the clay model indicated the limit to which some of Mr. Jurgenson's assumptions are valid. These assumptions include neglect of the restraining effect of the foundation material beyond the toes of the embankment, and the supposition that the surface between embankment and foundation remains plane during loading. It can be seen almost intuitively that variation in the ratio of $\frac{a}{L}$ will vary the extent to which these simplifying assumptions affect the applicability of Equation (9) to embankment analysis. The work with the clay model indicates that, for ratios of $\frac{a}{L} = 0.1$, Jurgenson's method and Haines' method are in substantial agreement as to the magnitude of the shearing stress induced by a given intensity of embankment loading. For higher values of the ratio, however, a considerable divergence between the two methods was noted. It was also observed that when the depth term, a, is small compared with L, the condition of simultaneous shear failure on an infinite number of surfaces in the plastic material is approached more closely than when a is comparatively large. In the latter case, evidence was obtained to indicate that shear failure occurs at a single point and, therefore, in giving the intensity of stress at this point, Jurgenson's formula (Equation (9)) does not indicate the full extent to which the foundation can carry load, due to the redistribution of stress following point failure. As against this so-called "point failure stress", Haines' method is believed to show the total resistance of the underground, or what might be termed the "total failure stress".

The argument just presented applies also in the case of the previously mentioned gelatin model-photo-elastic method. This method, except when

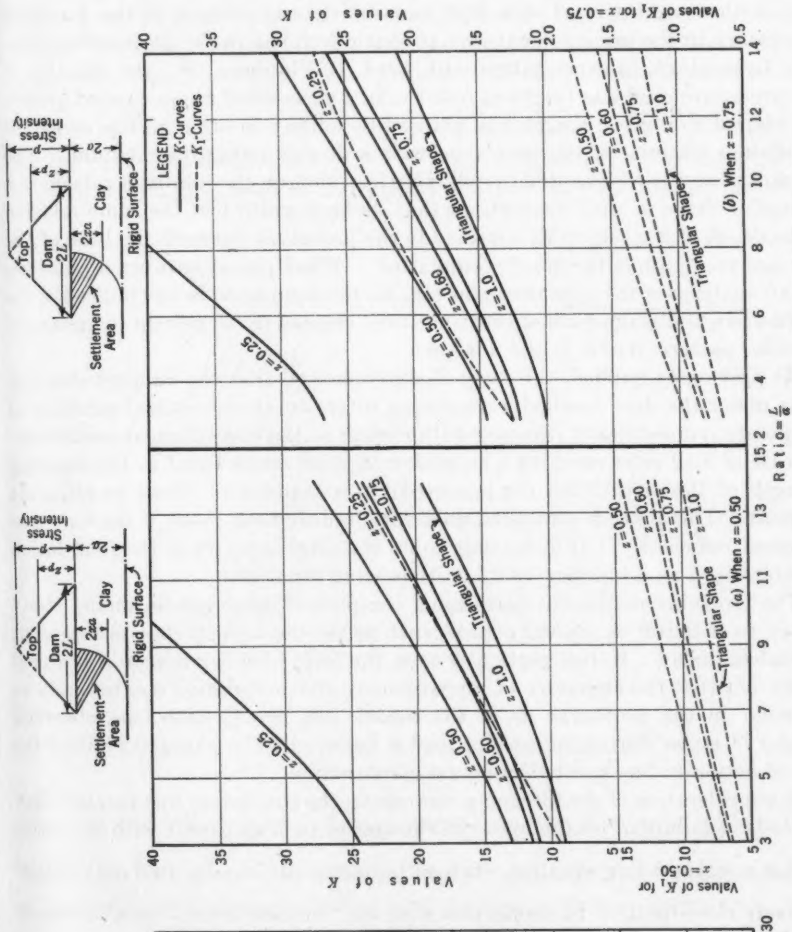


FIG. 40.—CURVE FOR SOLUTION OF SETTLEMENT FORMULA, EQUATION (8)

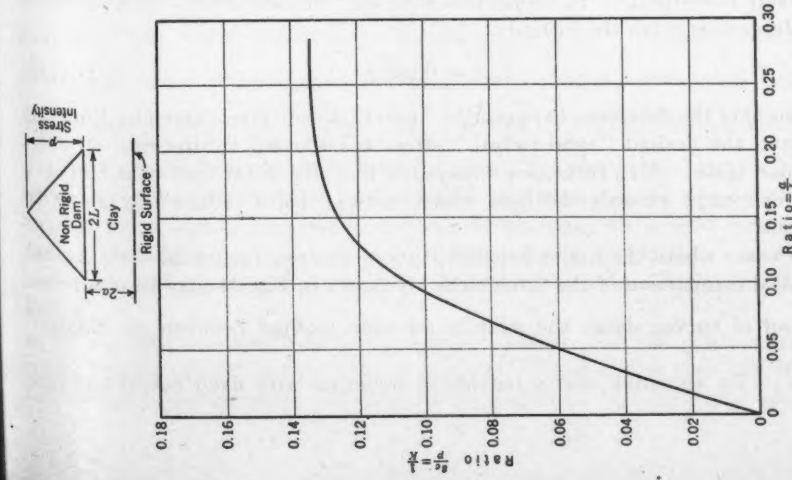


FIG. 39.—CURVE FOR SOLUTION OF INITIAL STABILITY FORMULA, EQUATION (7)

used in the qualitative studies first mentioned, was utilized in the Eastport Laboratory in the same general way as that developed in the Zanesville, Ohio, Soils Laboratory, in consultation with Fred. L. Plummer, M. Am. Soc. C. E. The procedure used was briefly as follows: In a glass-sided flume, melted gelatin was poured to a given height and allowed to solidify in order to represent the foundation material of the prototype with a doubly refracting medium. The embankment was represented by piling up lead shot on the solidified gelatin in a triangular shape of such proportions that its base width had the same relation to the depth of the gelatin as obtained in the prototype between the base of the dam and the depth of the plastic foundation. When placed between a polarizer and an analyzer of the type shown in Fig. 33, the completed model indicated the distribution and magnitude of shearing stress created in the gelatin by means of the color pattern visible in the analyzer.

As previously implied, this type of study showed that the induced shearing stress under the dam reached a maximum intensity at one central point or at two points symmetrically disposed with respect to the embankment center line. Increase of load after reaching a condition of point stress equal to the shearing strength of the gelatin led (as expected) to a transfer of stress to adjacent material. This process continued until total failure took place, if the load was increased sufficiently, but at no time could the total capacity of the foundation be determined as a function of its unit shearing strength.

The foregoing method of performing the photo-elastic-gelatin model study is open to criticism on several counts, such as the use of lead shot to represent the embankment. In this particular case, the force of this criticism is lessened by the fact that the character of embankment and foundation was actually as dissimilar in the prototype as in the model, but in any case, the observed transfer of stress during increase of load is believed to be characteristic of the type of action to be expected in actual construction.

In consideration of the foregoing comments, the conclusion was reached that the mode of failure of plastic material changes to such an extent with the ratio, $\frac{a}{L}$, that a straight-line equation, such as Equation (9), can be used only within relatively close limits. In connection with his "infinite depth" case for which Mr. Jurgenson gives the formula,

$$s = 0.256\,p \qquad (10)$$

the extent of the difference between the "point failure" stress given by Equation (10) and the desired "total failure" stress is indicated by the case of a rectangular plate. Mr. Jurgenson computes that the total load which such a plate can carry exceeds the load which causes "point failure" by about 80 per cent.

In cases where the major consideration is analysis for initial stability, the graphical comparison of the three methods shown in Fig. 41 may be of interest. This set of curves shows the relation for each method between the ratios, $\frac{s}{p}$ and $\frac{a}{L}$. By assuming that a particular structure with fixed height and base

width is being considered, this plot gives, directly, the variation of the stress induced by such a structure if built on clay deposits of different depths. Jurgenson's "rigid boundary" case plots as a 45° line, of course, until it intersects his "infinite depth" case at a value of $\frac{a}{L} = 0.256$. From this intersection the curve is a horizontal line. The work with the gelatin model plots as a curve showing a more gradual transition from the "rigid boundary" case to the

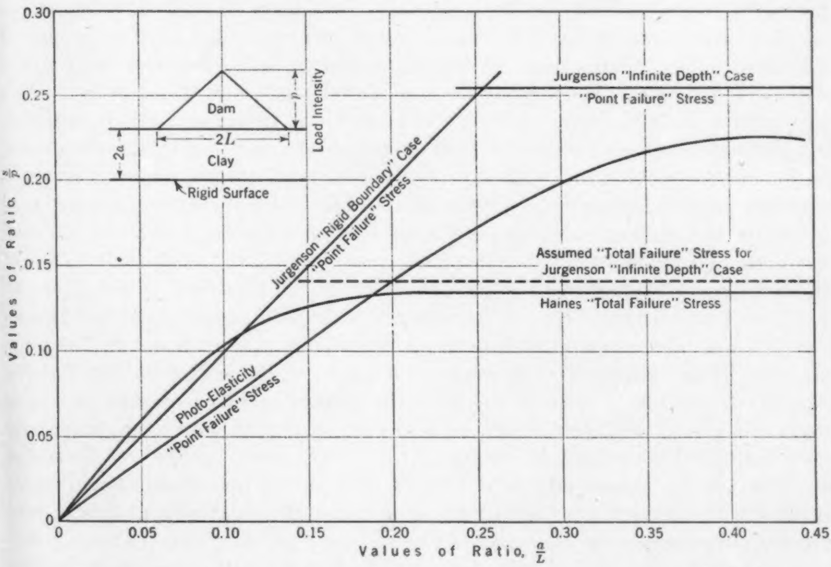


FIG. 41.—LOAD, SHEARING STRESS CURVES FOR TRIANGULAR EMBANKMENTS; $\frac{L}{a} = 4$.

"infinite depth" case. The curve for Haines' method of analyzing the stability of triangular embankment sections may be seen to follow Jurgenson's "rigid boundary" case quite closely up to the ratio, $\frac{a}{L} = 0.10$. Beyond $\frac{a}{L} = 0.15$, it deviates abruptly, however, and flattens out into what amounts to an "infinite depth" case at a much lower value of $\frac{s}{p}$. Since Haines' method is intended to give the "total failure" stress throughout, this is not surprising, however. If it is admitted that Jurgenson's "infinite depth" case has a factor of safety in the order of magnitude of 1.8, as indicated by the example of the rigid rectangular plate, a correction could be made, as shown by the dotted line. It is very interesting and significant to note how close this line is to that for the Haines method.

In the field in which Mr. Jurgenson's method and the photo-elastic—gelatin-model method have previously served, it is believed that the Haines method offers the advantage of being more general in the sense that it is not

confined to use in cases in which $\frac{a}{L} = 0.1$ and that it is more consistent in the sense that it always gives the "total failure" stress as previously defined.

The particular claim made for the Haines method, however, is that it can be extended beyond the field of usefulness of the other types. It is believed that work similar to that reported herein may attract the attention of engineers to the possibility of effecting economy in earth construction work by allowing artificial fills to sink into relatively soft and shallow foundations, rather than trying to attain stability by flattening the side slopes of such structures. In cases where a real economy is indicated by adopting this plan, as was the case at Passamaquoddy, the methods of Jurgenson and the gelatin-model study are of little, if any, value. The method developed by Haines, however, can be used for just such cases and as far as the writer knows, is the only method available for complete analysis for proposed earth embankments, since by its use investigation of the stability of such structures during all stages of construction is possible, and prediction of the extent of settlement which will occur due to shear failure of the underground may be made if such settlement is to be allowed. Estimate of settlement of this nature, of course, is essential to the calculation of the total cost of structures in which subsidence is anticipated.

It will be observed that in this paper much of the detail of the Haines analysis has been omitted although full particulars are given in the Passamaquoddy Tidal Power Development "Report of Soils Laboratory" dated September 1, 1936. This is due to a realization of the fact that the basic experimentation was performed with the objective of obtaining qualitative results applicable to one rather unusual type of problem. Since curtailment of activity on the Passamaquoddy Project took effect before all the intended studies of this nature were completed, it is considered advisable at this time to present only the general character of this research. It is hoped that this rather non-technical presentation will stimulate discussion and further study leading eventually to a complete and satisfactory solution of the indicated problem.

Summary

In connection with cohesionless, uncompacted earth or rock-fill embankments founded on cohesive soil, model studies indicate that embankment settlement resulting from shear failure and lateral displacement of the foundation material will take the form of the so-called "heart shape". By citing the specific case of the two large dams proposed for the Passamaquoddy Project, attention is invited to the possibility in some instances of effecting economy in embankment design by allowing over-stress of foundations to occur rather than flattening side slopes in an effort to prevent over-stress to such an extent that excess material is required.

The need for development of a comprehensive method of stress and settlement analysis for earth and rock-fill dams is indicated by a discussion of some available methods, and the so-called Haines method is outlined in general terms as a lead to further study and consideration of this subject.

ACKNOWLEDGMENTS

The investigations described herein were pursued as part of the study of the Passamaquoddy Tidal Power Project, under the direction of the Eastport, Me., District of the U. S. Corps of Engineers; Maj. Gen. E. M. Markham, U. S. Army, Chief of Engineers, U. S. Army, M. Am. Soc. C. E.; Lieut. Col. Philip B. Fleming, U. S. Corps of Engineers, was District Engineer, with Captain Hugh J. Casey, U. S. Corps of Engineers, M. Am. Soc. C. E., as Chief of the Engineering Division; John Sweeney, M. Am. Soc. C. E., was in charge of field exploration; and the writer, assisted by R. M. Haines, Jun. Am. Soc. C. E., conducted the Soils Laboratory studies.

SETTLEMENT OF STRUCTURES IN EUROPE AND METHODS OF OBSERVATIONS

By Charles Terzaghi,[23] M. Am. Soc. C. E.

Synopsis

The methods and results of settlement observations on several structures in Europe, selected to illustrate accurately, the behavior of structures on various types of compressible ground, are described in this paper. Particularly, emphasis is given to the observations of settlements of buildings on pile foundations, and the comparison between the settlement of individual piles under load tests with that of the entire pile foundation.

The form of the settlement diagrams with ground conditions and loadings is recommended as a pattern to be followed. The instruments and bench-marks used for settlement observations are described, but other instruments and methods of observation will suggest themselves to engineers. However, such methods should permit the observation of very small settlements as they are valuable in computing future settlements.

The observations are the result of studies extending over a 10-yr period; they were supervised by the writer and supported by contributions made by the Committee of the Society on Earths and Foundations.

Purpose of Investigations

The design of shallow foundations is governed by the condition that the greatest pressure exerted on the soil should not exceed a certain value, called the allowable soil pressure. If the structure is to rest on a pile foundation, the load per pile should not exceed the "safe load." Both types of foundations are subject to settlement. If the loaded stratum consists exclusively of sand or gravel, the settlement ceases within a few weeks or months after construction is finished. In this case the settlement can be expressed by a single set of recorded data or by a contour map showing the lines of equal settlement representing the subsidence of the loaded area after the settlement stopped. On the other hand, if the settlement is due to an increase of the load on a bed of clay, it can be recorded only by time-settlement curves, or by means of a set of contour maps showing the settlement at different times after construction was finished.

At present, there is no method of predicting the settlement of buildings on a sand or gravel foundation and the prospects for discovering such a method are

Note.—The Committee of the Society on Earths and Foundations, selected the settlement of structures as one of several for study and research. This paper, reporting an investigation sponsored by the Committee, was submitted to the Committee by its author, and the Committee has recommended its publication.

[23] Dr. Ing.; Prof., Technische Hochschule, Vienna, Austria.

very slight. No method of computing settlement can be accepted unless its validity is demonstrated by numerous settlement records obtained under very different practical conditions and no such collection of records has ever been presented. Current procedures for predicting the settlement of buildings on clay foundations are more promising; and, yet, according to reports,[24] the application of these methods requires elaborate investigations and is limited to important structures. In every other case the engineer must estimate the settlements on the basis of previous experience. Such experience can be acquired only by systematic settlement observations. Considering the present state of knowledge in the field of foundations, the settlement observations represent by far the most important type of research. The results of theoretical and of laboratory investigations in this field cannot claim more than an academic value until the importance of the errors have been investigated thoroughly and repeatedly. Any proposal for the practical application of a new method should include not only a detailed statement of several settlement forecasts, but also the results of adequate settlement observations, including the magnitude, the distribution, and the time rate of the settlement for the full-sized structures.

Method of Observation

In his earlier attempts to collect settlement data, the writer met with the following difficulties: (*a*) Most of the reference points (which consisted of bolts embedded in the masonry) disappeared during or immediately after construction; (*b*) in buildings with numerous partition walls the measurements within the building were difficult and not sufficiently accurate; and finally (*c*) the number of observation points was not great enough to permit the reliable construction of curves of equal settlement. In order to eliminate these difficulties the writer devised a new type of reference point shown in Fig. 42. It consists of a short piece of pipe, which is entirely embedded in the masonry (Fig. 42(*a*)). Normally, the opening of this pipe is closed by means of a brass plug, the outer surface being flush with the wall. In order to make a settlement observation, the plug is removed and temporarily replaced by a cylindrical piece such as that shown in Fig. 42(*b*). The observations are made not with a level, but by means of the device shown in Fig. 42(*c*) which consists of two glass tubes connected by a rubber hose. The entire system is filled with water, and in each tube the position of the water level is measured by means of the micrometer screw shown in Fig. 42(*d*). The vertical distances, Z_0, in Fig. 42(*c*) are always the same for both tubes, and the distances, Z_1 and Z_2, are measured by means of the micrometer screw. Hence, the difference between the elevation of the two reference points is equal to $\Delta Z = Z_1 - Z_2$. A careful survey of the errors of observation performed by Professor H. Löschner,[25] in Brunn, Austria, has shown that the error does not exceed 0.05 mm, or 0.002 in.

In order to obtain a reliable conception of the distribution of the settlement over the area occupied by the building, at least one reference point should be established for each 200 sq ft of this area. At least, one-third of all the points should be within the building. All the recent settlement data embodied in the following paragraphs were obtained by means of the device shown in Fig. 42.

[24] *Proceedings*, Conference on Soil Mechanics, 1936.

[25] "Genauigkeitsuntersuchung zur Messung von Setzungen nach dem Verfahren von Professor Terzaghi," by H. Löschner, *Zeitschrift für Instrumentenkunde*, Vol. 56, July, 1936, Heft 4.

THE BENCH-MARK

In order to obtain reliable information as to the downward movement of a structure, the levels must be referred to a bench-mark that does not move. Usually, this bench-mark is established on some existing building at a distance of 100 or 200 ft from the structure under observation. If no reliable settlement record is available for this building, at least two bench-marks should be established on buildings, located on different sides of the structure under observation.

Fig. 43 shows the curves of equal settlement of a group of factory buildings for the year 1934. Construction was finished in 1929 and the observations

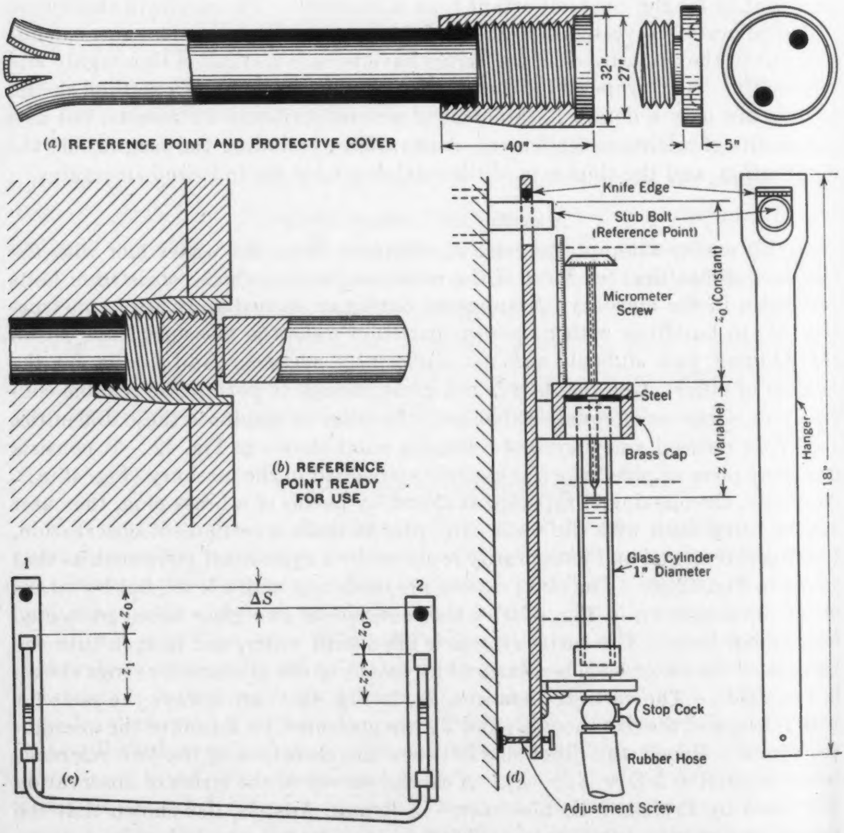


FIG. 42.—DEVICE FOR ACCURATE MEASUREMENT OF SETTLEMENT

were begun early in 1930. The values that appear within the areas occupied by the buildings represent the extreme limits for the load, in tons per square foot of the area, after construction was finished. The buildings are situated on a stratum of very stiff tertiary clay, more than 600 ft thick, and the bench-mark was established in a limestone quarry in the vicinity of the plant. The compressibility of the clay decreases to a depth of about 25 ft below the surface,

and the distance between the individual buildings is considerable. Nevertheless, the settlement extends over the entire area occupied by the plant. The

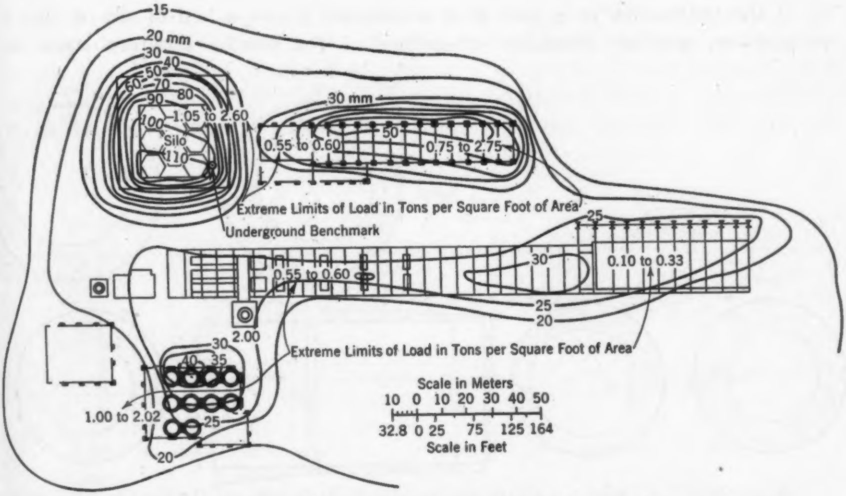


FIG. 43.—OBSERVED CURVES OF EQUAL SETTLEMENT AFTER 4.5 YEARS; GROUP OF FACTORY BUILDINGS CONSTRUCTED IN 1929 (OBSERVATIONS BEGAN IN 1930)

contour interval in Fig. 43 is 5 and 10 mm, the equivalent values, in inches, being as follows:

Millimeters	Inches	Millimeters	Inches
15	0.59	60	2.36
20	0.79	70	2.76
25	0.98	80	3.15
30	1.18	90	3.54
40	1.57	100	3.94
50	1.97	110	4.33

Fig. 44 represents the settlement of a group of oil tanks.[26] At each point of the circumference of the buildings the radial width of the shaded area represents the settlement. The diagram shows plainly the influence of the weight of each structure on the settlement of adjacent structures, particularly the units toward the right. The 1932 Progress Report of the Committee on Earths and Foundations contains a brief mention of this case, including a section through the soil.[27]

Due to the inevitable subsidence in the vicinity of the loaded area, the short distance between the bench-mark (Fig. 42) should never be nearer the building under observation than twice the width of the building. If this distance exceeds 100 ft, it is advisable to establish intermediate points of observation. Each set of levels should be carried back to the point where it began and the settlement record should contain the ultimate error. With few exceptions, the

[26] "Kritische Betrachtung von Flach-und-Pfahlgründungen," von W. Loos, Berlin, Julius Springer, 1932.

[27] *Proceedings*, Am. Soc. C. E., May, 1933, p. 777.

observations made by the writer using the device shown in Fig. 42 have been made in rather inaccessible cellars with many partition walls; and yet the ultimate error has never exceeded 0.02 in.

If the entire city or a part of it is situated above a bed of silt or clay a progressive, general, irregular subsidence of the loaded territory must be

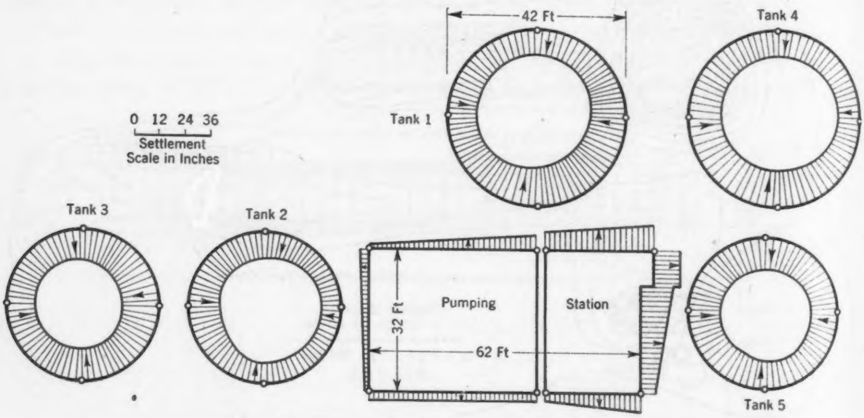


FIG. 44.—SETTLEMENT OF A GROUP OF OIL TANKS

expected. In Cambridge, Mass., the entire area on both sides of Massachusetts Avenue, settled from 1869 to 1909 (a period of 40 yr) through distances ranging between zero and 2 ft.[28] A similar general subsidence was found to have occurred in San Francisco, Calif., in the district adjoining Market Street. From 1923 to 1928, the greatest subsidence was more than 0.2 ft.[29] There is but little doubt, that a similar general and irregular subsidence occurs in every city built upon beds of clay or silt. If such a general subsidence occurs the foregoing procedure yields data that inform the engineer merely as to the difference between the general downward movement and the downward movement of the newly constructed building. In order to obtain the actual settlement it would be necessary to establish a bench-mark at the bottom of a drill hole which extends through the entire system of unconsolidated strata to bed-rock. Such a bench-mark was established in Cambridge, Mass., at a depth of about 125 ft below the surface. Such bench-marks would also facilitate the collection of data concerning the general subsidence of the area occupied by a city. It should be the duty of municipal engineering departments to keep records of such movements.

SETTLEMENT OF PILE FOUNDATIONS

Observations made in Vienna, Austria, refer exclusively to foundations on conical, cast-in-place piles with a length of about 20 ft. They are arranged in two or three rows beneath continuous footings. The top-soil stratum usually consists of a loose, loamy, artificial fill and the piles get their support in a

[28] "Boston Foundations," by J. R. Worcester, M. Am. Soc. C. E., *Journal*, Boston Soc. of Civ. Engrs., January, 1914.

[29] "Subsidence and the Foundation Problem in San Francisco," San Francisco Section, Am. Soc. C. E., September, 1932.

stratum of firm sand and gravel or very stiff clay of great thickness. The geological conditions practically exclude the presence of soft strata at any depth below the points of the piles. Before the settlement observations were made, many experienced contractors of the city believed that the settlement of the pile foundations was approximately equal to that produced by a loading test on an individual pile. According to this view, if all the piles are assigned equal loads, the settlement of the entire foundation should be uniform. Observations made during the past few years have destroyed both illusions. Figs. 45, 46,

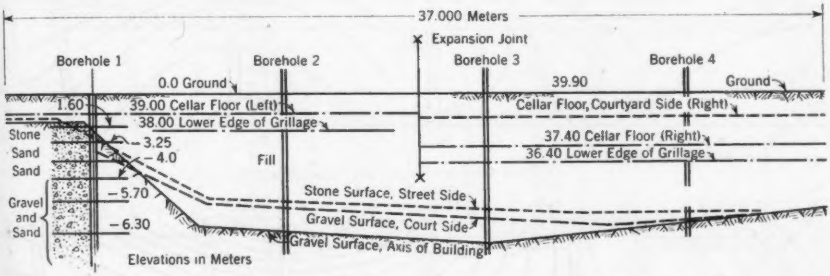


FIG. 45.—SOIL CONDITION AT SITE OF A BRICK BUILDING, IN VIENNA, AUSTRIA

and 47(*a*) illustrate an example of such observation. The building consists of brick walls with continuous footings, 40 in. wide, supported by two rows of conical piles. Each pile carries a load of about 24 tons. The upper curve in Fig. 46(*b*) shows the result of a loading test on an individual pile, located at Point *P* in Fig. 47(*a*). Under a load of 24 tons a conical pile in Vienna very seldom settles more than a fraction of a millimeter during the loading test, and the loading test on the pile at Point *P* merely confirmed this empirical rule; and yet the settlement of the entire pile foundation was equal to more than forty times the settlement of the individual pile carrying the same load during

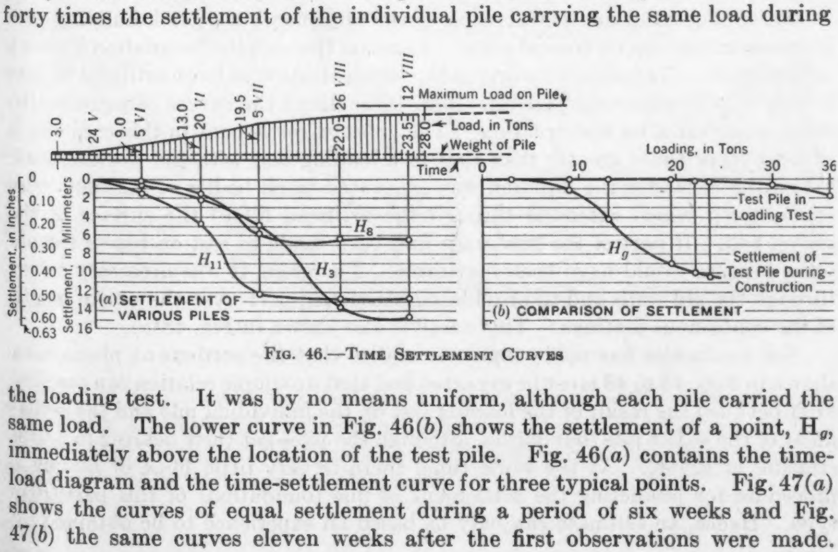


FIG. 46.—TIME SETTLEMENT CURVES

the loading test. It was by no means uniform, although each pile carried the same load. The lower curve in Fig. 46(*b*) shows the settlement of a point, H_g, immediately above the location of the test pile. Fig. 46(*a*) contains the time-load diagram and the time-settlement curve for three typical points. Fig. 47(*a*) shows the curves of equal settlement during a period of six weeks and Fig. 47(*b*) the same curves eleven weeks after the first observations were made.

Since the time-settlement curves for various parts of the building are different the shape of the curves of equal settlement changes although the distribution of the load over the loaded area remains practically unaltered.

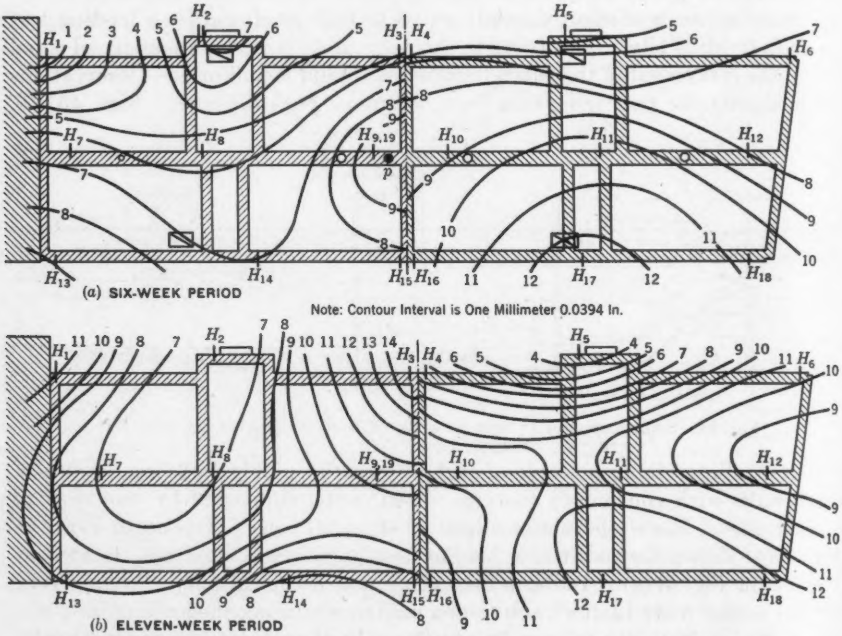


FIG. 47.—CURVES OF EQUAL SETTLEMENT: (a) SIX-WEEK PERIOD; (b) ELEVEN-WEEK PERIOD

Fig. 48 illustrates another example of the settlement of a brick building with continuous footings on conical piles. A section through the foundation is shown in Fig. 48(*a*). The piles were driven through a stratum of loose artificial fill into a layer of well compacted gravel. Fig. 48(*b*) shows the curves of equal settlement, one year after construction. The greatest settlement in this case also is at least forty times greater than that in a loading test with the working load. When the cellar for the building was excavated thick bodies of masonry were encountered which extended through the artificial fill to the surface of the gravel bed. If part of the new walls had been made to rest on top of the old ones, rupture would have been inevitable. Therefore, it was necessary to cut through the old walls and to provide an elastic support over the entire length of the continuous footings. This detail is also shown in Fig. 48(*a*).

Soil mechanics has made engineers realize that the settlement phenomena shown in Figs. 45 to 48 must be expected and that no simple relation can possibly exist between the result of the loading test on the individual pile and the settlement of the entire pile foundation, although the piles get their bearing in a firm stratum of gravel. At the same time, there is very little hope of finding a procedure for predicting the settlement of pile foundations of this particular type. Hence, an estimate can only be based on experience to be obtained by

systematic observations on foundations supported by piles, with different lengths, driven to refusal in different types of strata. No general conclusions can be derived from two or three sets of observations. Thus, in both cases, represented by Figs. 45 to 48, the settlements ended not later than about two

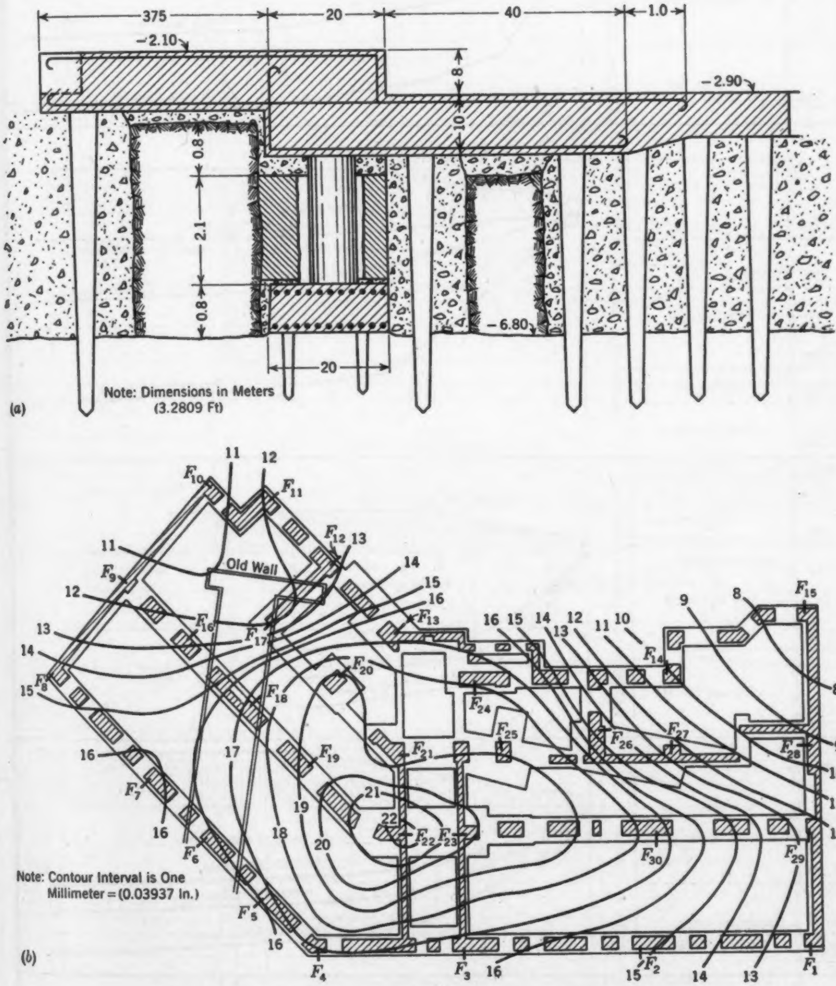
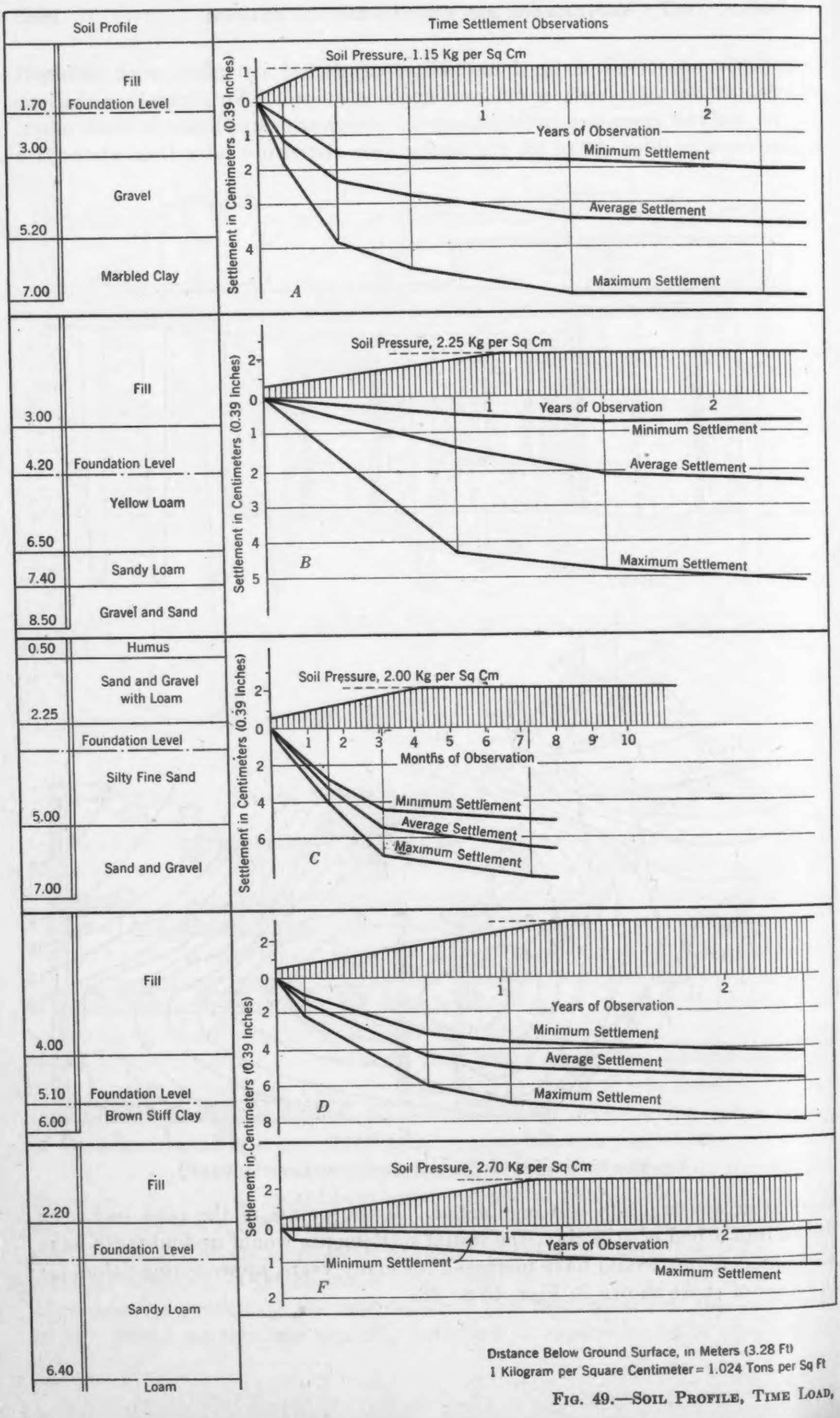


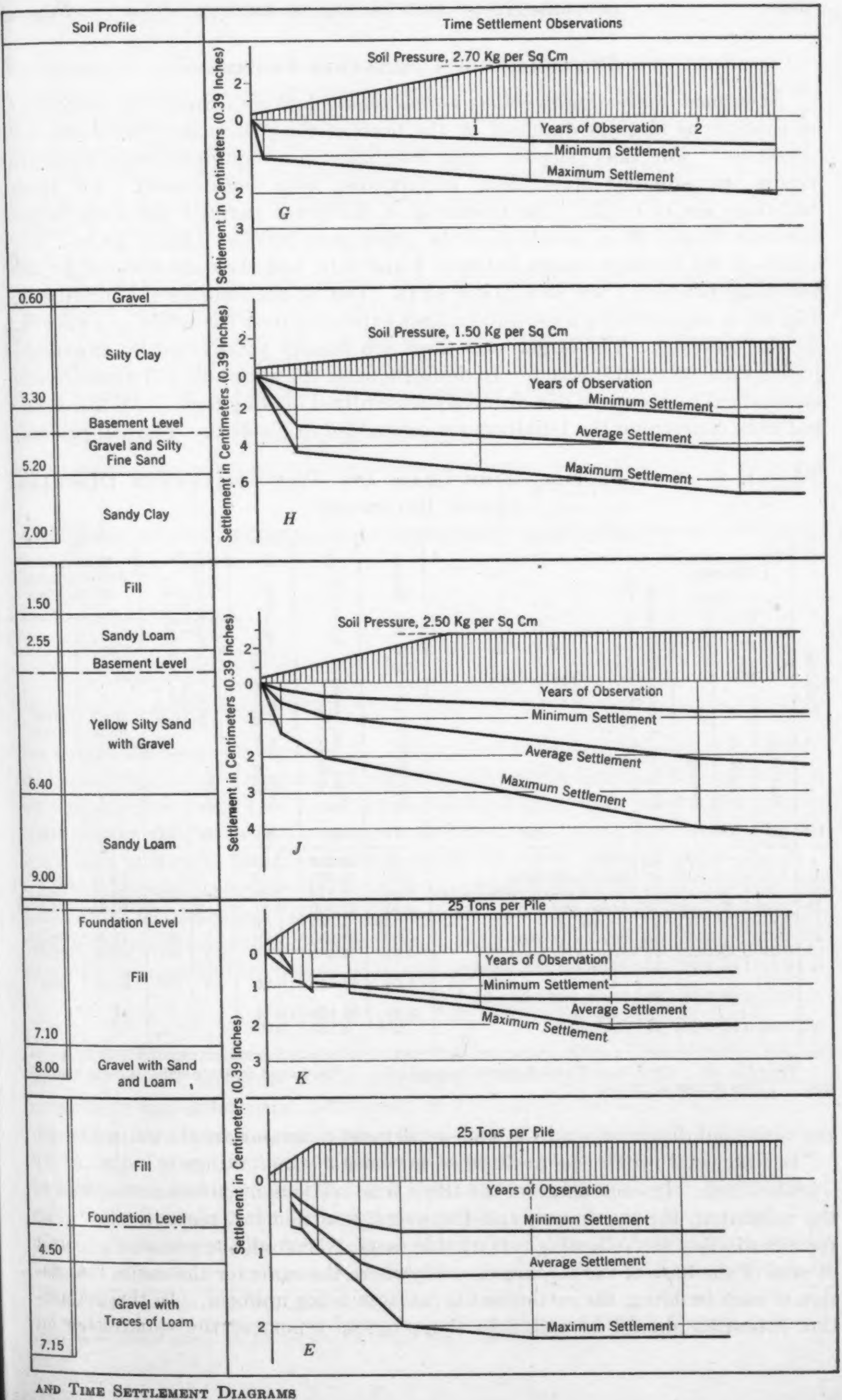
FIG. 48.—SETTLEMENT OF BRICK BUILDING ON CONICAL PILES

years after construction was completed. If the points of the piles had been driven into a bed of stiff clay, the initial settlements would undoubtedly have been smaller, but would have increased for many years, approaching values far in excess of those shown in Figs. 45 to 48.



Distance Below Ground Surface, in Meters (3.28 Ft)
1 Kilogram per Square Centimeter = 1.024 Tons per Sq Ft

FIG. 49.—SOIL PROFILE, TIME LOAD,



AND TIME SETTLEMENT DIAGRAMS

Settlement of Continuous Footings

It seemed interesting to collect at least some data concerning the settlement of continuous footings designed on the basis of the customary "allowable soil pressure." For that purpose eight buildings were equipped with reference points immediately after their foundations were constructed. All these buildings are of brick. The thickness of the lower parts of the walls ranges between 20 and 30 in. and that of the upper part between 15 and 20 in. The width of the footings ranges between 2 and 9 ft, and they are covered by the buildings between 4 000 and 20 000 sq ft. One of the structures (Building *A*, Fig. 49) is supported by a continuous slab, extending over the entire area covered by the building. The larger structures are usually subdivided by expansion joints, from 50 to 80 ft apart. In doubtful cases the allowable soil pressure was determined by means of one or more standardized loading tests. All the essential data concerning the buildings are assembled in Table 6. The soil profiles,

TABLE 6.—Soil Profile, Time Load, and Time-Settlement Diagrams (Brick Buildings)

Building	Settlement, in Inches			Number of observation points	Type of soil *	Soil pressure, in tons per square foot	Total area covered by building, in square feet	Load per unit of total area, in tons per square foot	Allowable Soil Pressure Determined by Loading Tests, in Tons per Square Foot		Width of Foundations, in Inches	
	Maximum	Minimum	Average						From:	To:	Minimum	Maximum
(1)	(2)	(3)	(4)	(5)	(6)	(7)	(8)	(9)	(10)	(11)	(12)	(13)
A	2.11	0.71	1.26	60	Fill	1.07	20 300	1.07	1.1	1.9	‡	....
B	2.11	0.30	0.99	27	Sand with loam	2.09	15 300	0.84	2.3		53.2§	90.6
C	2.76	1.73	2.16	21	Silty fine sand with loam	1.86	3 910	0.84	1.4	1.5	50.0§	88.5
D	2.34	1.87	2.05	48	Loam and clay	2.51	12 100	1.41	2.3		49.2§	74.8
E	0.87	0.28	0.55	31	Gravel fill	25†	5 360	1.02	..	..	||	....
F	0.51	0.12	0.36	33	Sand with loam	2.32	10 200	0.90	3.7	4.7	47.3§	98.5
G	0.83	0.30	0.55	43	Sand with loam	2.51	6 480	0.93	3.7	4.7	43.4§	67.0
H	2.64	1.06	1.77	40	Marbled, stiff, plastic clay	1.63	16 640	0.96	2.3	3.7	47.3§	82.6
J	1.77	0.36	0.99	180	Loam, and loam with gravel	2.32	91 100	0.84	..	..	§	....
K	0.96	0.26	0.55	19	Gravel fill	25†	3 720	0.93	..	..	||	....

* See Fig. 49. † Buildings *E* and *K* are in tons per pile. ‡ Reinforced concrete slab. § Strip foundation. || Piles driven to gravel.

the time-load diagrams, and the time-settlement diagrams are shown in Fig. 49.

In every soil profile the position of the base of the footings is indicated by a broken line. In each diagram the three time-settlement curves correspond to the minimum, the maximum, and the average settlement, respectively. The records disclose the following remarkable facts: Although the pressure per unit of area of the base of the footings is everywhere the same for the entire foundation of each building, the settlement is far from being uniform. In this connection attention should be called to the progress report of the Committee on

Earths and Foundations[30] which demonstrates that the settlement produced by a uniform load on a circular area is also far from being uniform. The ratio between the largest and the smallest settlement of the buildings listed in Table 6 ranges from 1.25 to about 7.00. The "allowable soil pressure" for the buildings listed in Table 6 was selected on the basis of a standardized procedure and the thickness of the highly compressible strata never exceeds about 20 ft. Nevertheless, the average, settlement ranged between the wide limits of 0.25 in. and more than 2 in.

In Germany, the foundations for bridge piers are also based on standardized "allowable soil pressures"; and yet since the bridges were built in very different parts of a large country, the thickness of the compressible strata and, as a consequence, the settlement of the structures, varies between considerably wider limits than it does in Vienna. Table 7 contains an abstract of data that have

TABLE 7.—SETTLEMENT OF BRIDGE PIERS IN GERMANY

Number of structures	Description of soil	Average Soil Pressure, in Tons per Square Foot		Settlement, in Inches	
		From:	To:	From:	To:
(1)	(2)	(3)	(4)	(5)	(6)
9	Silt	1.1	2.0	8	40
8	Clay, loess, loam, etc	1.1	2.6	2	8
31	Boulder clay, sand or gravel with clay	2.5	4.0	0	0.8
21	Sand, gravel	1.5	3.0	0	0.4

been collected by L. Casagrande.[31] The settlement ranges between almost zero and more than 3 ft. In general, the settlements of the foundations on sand and on boulder clay, were very small and stopped a short time after construction was completed, whereas the important settlement of foundations on clay and silt continued and must be expected to increase for many years to come.

The preceding statements have demonstrated that the limitation of soil pressure to the so-called "allowable" values by no means involves the limitation of the settlement to a definite value. Experience in estimating settlement can only be acquired by means of reference points and levels.

DIFFERENTIAL SETTLEMENT

If a building settles uniformly, no matter how much, the stresses in the members of the structure are not affected. Therefore, it was interesting to investigate the differential settlement of continuous footings which were designed on the basis of equal loads per pile, or of a uniform soil pressure. In Fig. 50(*a*), Curve 1 represents the settlement of the straight front wall of the building discussed in connection with Figs. 45 to 47; Curve 2, the settlement of the rear wall; and Curve 3, that of the middle wall.

Fig. 50(*b*) shows the distortion of the walls of the building represented by Fig. 48. Curve 1 corresponds to the front wall of the building, Curve 2, to

[30] *Proceedings*, Am. Soc. C. E., May, 1933, p. 812, Case F.

[31] "Setzungsbeobachtungen an Brückenbauten der Reichsautobahnen," von L. Casagrande, *Proceedings*, International Assoc. for Bridge and Structural Eng., Berlin, 1936.

the outside wall toward the court, and Curve 3 refers to the wall that runs parallel to the front wall through the interior of the building. In both cases (Figs. 50(*a*) and 50(*b*)) the continuous footings were supported by almost equidistant piles in such a fashion that every pile carried practically the same load.

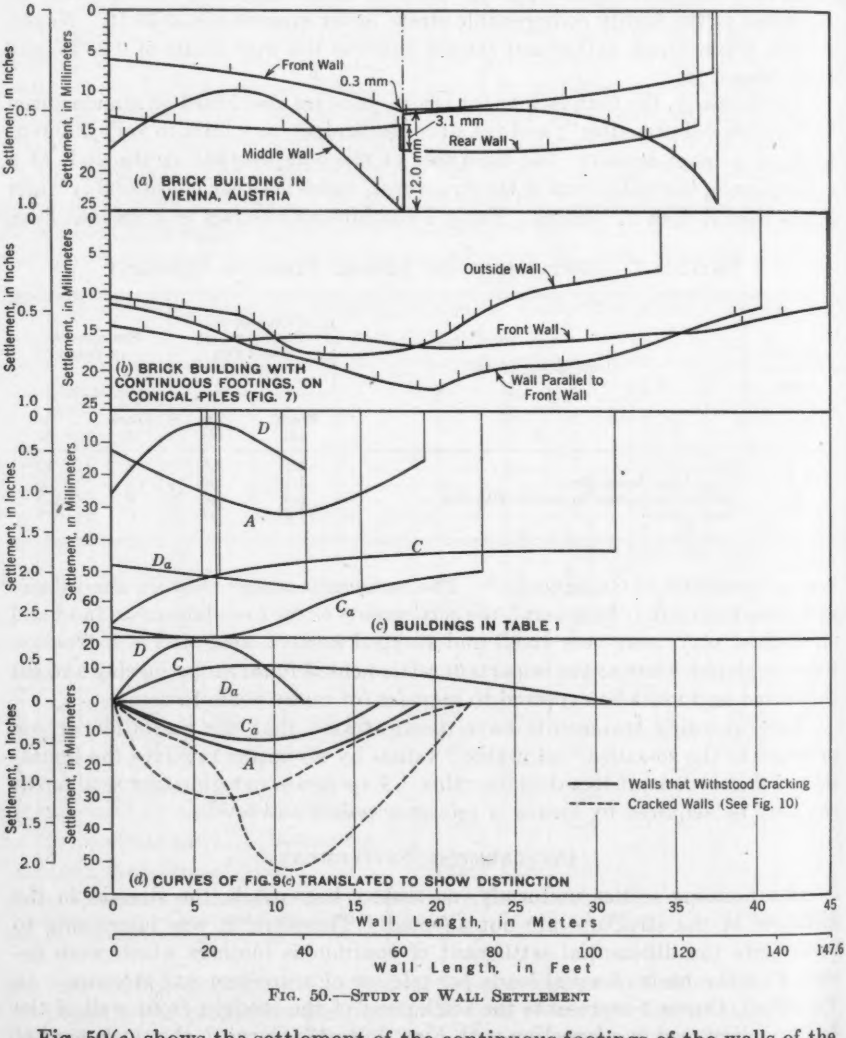


FIG. 50.—STUDY OF WALL SETTLEMENT

Fig. 50(*c*) shows the settlement of the continuous footings of the walls of the buildings listed in Table 6 and of several other buildings. These footings are supported by the natural ground without the assistance of piles, and they were designed in such a fashion that the pressure per unit of area of the footing is uniform throughout. Fig. 50(*d*) was obtained from Fig. 50(*c*) by translating the two ends of each curve up to the horizontal axis. As a consequence, Fig.

50(*d*) shows the distortion of the walls produced by differential settlement. All the walls represented by full black lines withstood the deformation without cracking. As a supplement to this diagram, two dotted curves were introduced. They represent the distortion of two of the walls of the building shown in Fig. 51 which is supported by a massive concrete slab, 4 ft thick. The distortion of these walls is greater than a brick wall can stand, as evidenced by the shearing cracks.

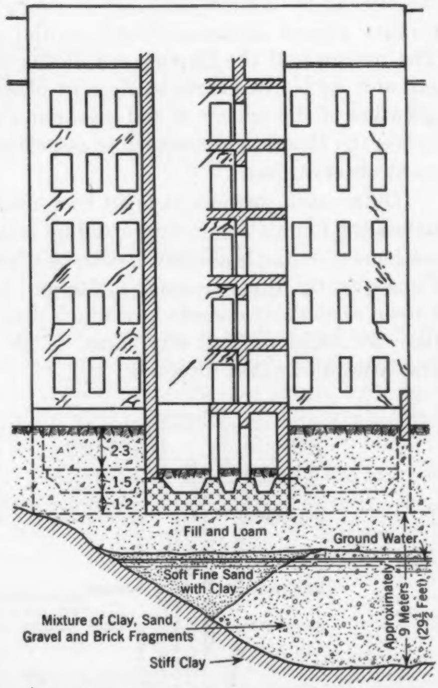


FIG. 51.—CRACKS IN BUILDING WALL DUE TO SETTLEMENT (SEE TABLE 6)

The facts represented in Fig. 50 lead to several interesting conclusions. Contrary to what many engineers still believe, settlement is never uniform, although the load per pile or the load per unit of area of the soil is the same everywhere. In none of the diagrams (except Fig. 48) do the curves of equal settlement have any resemblance to what one would expect from the theory of the settlement of loads supported by a homogeneous medium. Therefore, the differential settlement of the buildings represented by the diagrams is essentially due to local variations in the compressibility of the loaded strata. The importance of these variations cannot possibly be appraised by preliminary investigations of any kind, at a reasonable expense. Hence, an estimate of the differential settlement must be based on previous experience and this experience can only be acquired by systematic settlement observations such as those described herein. However, experience has shown that the erratic character of differential settlement is limited to those cases in which the thickness of the highly compressible strata does not exceed 20 to 25 ft. The greater the thickness of the highly compressible strata, the smaller becomes the difference between the real differential settlement and the settlement corresponding to a uniform and homogeneous medium.

For a given degree of non-uniformity of the subsoil, the importance of the differential settlement decreases with increasing stiffness of the walls. Thus, in the buildings discussed in connection with Figs. 45 and 48, the front wall (lower boundary of the area occupied by the building) is straight and the corresponding distortion, shown by Curve 1 to Fig. 50(*a*) is insignificant. On the other hand, the rigidity of the rear wall (upper boundary of the building lot in Figs. 45 and 48) is reduced by several corners. Therefore, the differential settlement shown

by Curve 2 in Fig. 50(*a*), is far more important. A comparison of Figs. 48 and 50(*b*) indicates similar relations between the rigidity of the walls and the importance of the differential settlement. The settlement of the stiff wall, 1, is far more uniform than that of Wall 3, which, in plan, represents a broken line.

Any theoretical computation of settlements is based on the assumption that the subsoil is uniform, at least in every horizontal direction. The non-uniformity always increases the differential settlement and its detrimental effects. The nature and the importance of the difference between theory and reality can only be learned from settlement observations. Therefore, a practical application of the science of soil mechanics to settlement problems is not feasible unless the theoretical research is associated with, and supplemented by, settlement observations.

Differential settlement must be considered inevitable for every foundation, unless the foundation is supported by solid rock. The effect of the differential settlement on the building depends to a large extent on the type of construction. Consequently, it is necessary to find out by careful inspection of buildings with known settlement records how much distortion the different types of construction can stand without any harm. The data shown in Fig. 50(*d*) represent a first attempt in that direction.

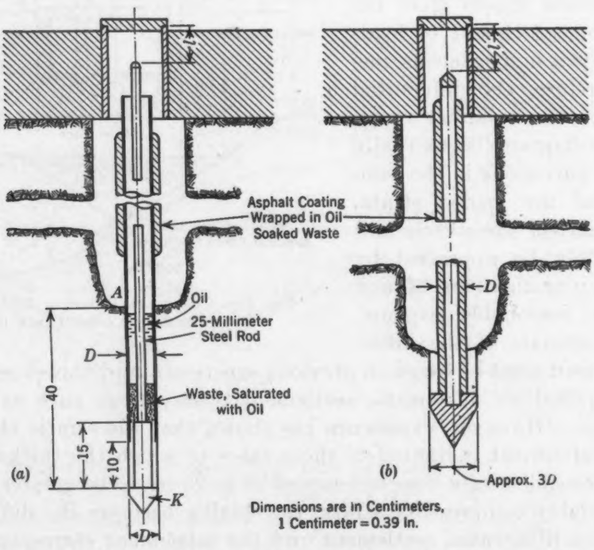


FIG. 52.—UNDERGROUND BENCH-MARK USED IN DETECTING THE COMPRESSION OF STRATA AT VARIOUS DEPTHS

UNDERGROUND BENCH-MARKS

If it is desired to keep the settlement of a structure within specified limits, the foundation must be carried at least to a certain minimum depth which depends, among other factors, on the nature and the thickness of the soil strata beneath the building site. Similarly, if the settlement of an existing building is to be stopped by underpinning, such underpinning must be extended to

some level below the seat of the settlement. The science of soil mechanics has furnished a set of theoretical rules for determining the distribution of the settlement over a vertical section through the loaded soil; and yet no such rules

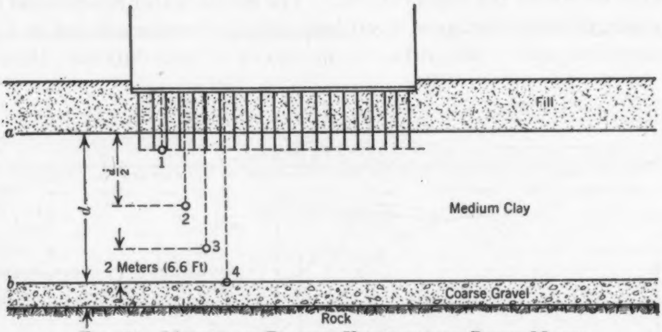


FIG. 53.—METHOD OF PLACING UNDERGROUND BENCH-MARKS

can be accepted for practical application on a broad scale until they have been confirmed by the results of observations on existing structures. Such observations can only be made by means of underground bench-marks; that is, by means of reference points established at the bottom of drill holes, at different depths below the surface of the ground.

A description of reference points of the type was presented by the writer[32] in 1930. Figs. 52 and 53 describe these reference points and show how to establish them for the purpose of ascertaining the seat of the settlement of a pile foundation above a bed of medium clay. A method for constructing such

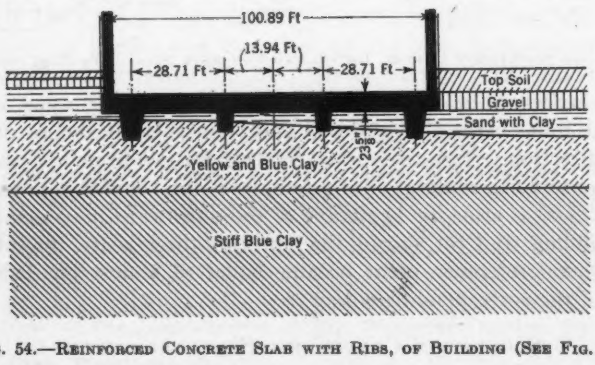


FIG. 54.—REINFORCED CONCRETE SLAB WITH RIBS, OF BUILDING (SEE FIG. 43)

bench-marks in very soft clay was described in 1936 and used in the City of Mexico, Mexico, by José A. Cuevas.[33] An underground bench-mark similar to that shown in Fig. 52(*b*) was established beneath one of the corners of the building marked "Silo" in Fig. 43, at a depth of 15 ft beneath the base of the foundation. As shown in Fig. 54, the building rests on a reinforced concrete slab strengthened by ribs. To a depth of about 8 ft below the base of the slab the soil consists of a yellowish, weathered, rather compressible clay. Beneath it

[32] "Die Tragfähigkeit von Pfahlgründungen," von Charles Terzaghi, *Die Technik*, 1930, Heft 31 und 34.
[33] *Proceedings*, International Conference on Soil Mechanics, Paper No. 5, Vol. 1, Cambridge, Mass., 1936.

is a very thick bed of stiff, blue clay. The underground reference point served the purpose of determining the extent to which the compression of the stiff blue clay was responsible for producing the total settlement of the structure. Fig. 55 shows the results of the observations. The excess water is squeezed from the clay, escaping through the layer of silt between the weathered clay and the base of the foundation slab. According to the theory of consolidation, the compres-

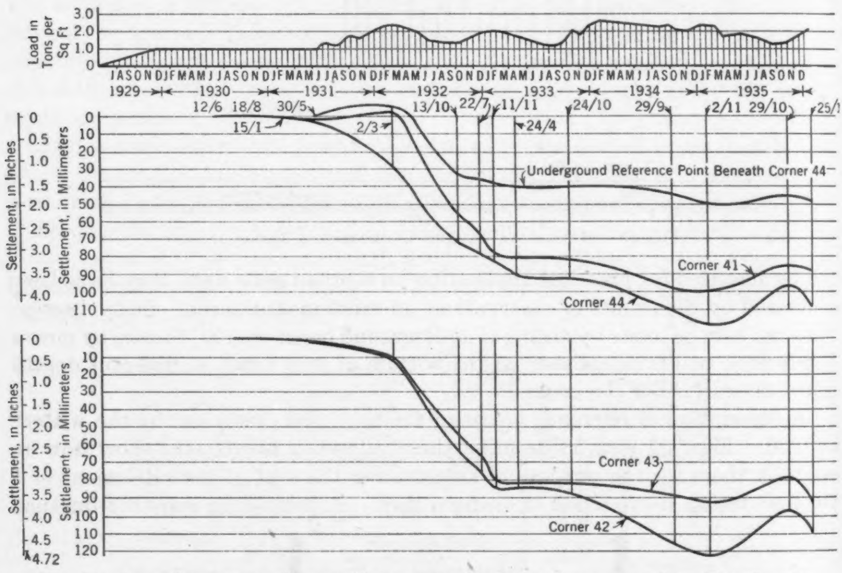


FIG. 55.—LOAD SETTLEMENT CURVES, REINFORCED CONCRETE SLAB, WITH RIBS (SEE FIG. 54)

sion of the clay would be expected to begin at the point of contact between the clay and the more permeable material and from there it should proceed in a downward direction. This theoretical forecast was confirmed by the fact that the settlement of the underground reference point beneath Corner 44 began about eight months after the beginning of the downward movement of Corner 44 of the building; and yet, during the following three years, the settlement of the underground reference point became equal to almost one-half the total settlement of the corresponding corner.

SUGGESTION CONCERNING STANDARDIZATION

No method of soil investigation and of settlement computation can be accepted for practical use until the degree of accuracy of the results has been determined by experience. Such experience can be secured only by reliable and adequate settlement observations. Therefore, the first and the most urgent task in the establishment of a science of soil mechanics consists in standardizing the procedure for making settlement observations.

AMERICAN SOCIETY OF CIVIL ENGINEERS

Founded November 5, 1852

DISCUSSIONS

PRACTICAL APPLICATION OF SOIL MECHANICS
A SYMPOSIUM

Discussion

BY MESSRS. GEORGE E. BEGGS, M. L. ENGER, R. J. FOGG, HARRY T. IMMERMAN, D. P. KRYNINE, F. A. MARSTON, GEORGE PAASWELL, RALPH R. PROCTOR, CHARLES TERZAGHI, LAZARUS WHITE, AND GLENNON GILBOY

GEORGE E. BEGGS,[34] M. L. ENGER,[35] R. J. FOGG,[36] HARRY T. IMMERMAN,[37] D. P. KRYNINE,[38] F. A. MARSTON,[39] GEORGE PASSWELL,[40] RALPH R. PROCTOR,[41] CHARLES TERZAGHI,[42] AND LAZARUS WHITE,[43] MEMBERS, AM. SOC. C. E., AND GLENNON GILBOY,[44] ASSOC. M. AM. SOC. C. E. (by letter).[44a]—Realizing that it is most important to obtain exact settlement data for existing structures, together with the underground conditions at the various sites, the Committee of the Society on Earths and Foundations has been financing, for the past years, an extended investigation by Charles Terzaghi, M. Am. Soc. C. E., mostly in Europe and, in part, also in the United States. These investigations have been valuable as a check on existing theories. In some cases quite

NOTE.—This Symposium is published in this number of *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the Symposium. The writers of this discussion comprised the membership of the Committee of the Society on Earths and Foundations (Mr. White, *Chairman*), which was authorized by the Board of Direction on October 10, 1927. In January, 1937, the Committee and its activities were transferred, and re-allotted, logically, among the various Committees of the newly authorized Soil Mechanics and Foundations Division.

[34] Prof., Civ. Eng., Princeton Univ., Princeton, N. J.

[35] Dean, Coll. of Eng., and Director of Eng. Experiment Station, Univ. of Illinois, Urbana, Ill.

[36] Cons. Engr., Washington, D. C.

[37] Chf. Engr., Spencer, White & Prentis, Inc., New York, N. Y.

[38] Research Associate in Soil Mechanics, Dept. of Civ. Eng., Yale Univ., New Haven, Conn.

[39] Cons. Engr. (Metcalf & Eddy), Boston, Mass.

[40] Mgr., Spencer, White & Prentis, Inc., Detroit, Mich.

[41] Field Engr., Bureau of Water-Works and Supply, City of Los Angeles, Los Angeles, Calif.

[42] Dr. Ing.; Prof., Technische Hochschule, Vienna, Austria.

[43] Pres., Spencer, White & Prentis, Inc., New York, N. Y.

[44] Assoc. Prof., Soil Mechanics, Mass. Inst. Tech., Cambridge, Mass.

[44a] Received by the Secretary December 12, 1936.

close agreement has been found between computed and observed settlement, by the use of the Boussinesq equations to obtain internal pressures below the footing, and Professor Terzaghi's equations of consolidation as determined by compression based on undisturbed samples.

Professor Terzaghi has devised a graphic form of settlement curve and a diagrammatic method indicating conditions in the foundation and underground, which is compact and complete. He calls the compressible strata beneath the settling structure the "seat of settlement" and the shape assumed by the base of the structure "the settlement trough". He calls attention to the over-emphasis previously made on the pressure at the base of the footings; and he demonstrates by observation and calculation that most of the observed settlements are due to compression of the unconsolidated materials within the mass of the supporting underground; and that the "seat of settlement" may extend hundreds of feet in depth.[45]

At the request of the writers, Professor Terzaghi has kindly prepared a summary of his work on settlements with particular attention to the methods of obtaining accurate observations, and giving typical cases with a non-mathematical discussion.[46] Particular attention is called to the failure to obtain uniform settlements, even when uniform loading of walls, etc., was closely approximated, both in the case of pile foundations and spread footings.

The writers believe that this contribution is of great value. They hope that it will stimulate discussion, and are particularly desirous that other cases of observed settlements be submitted, with complete data.

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[45] *The Structural Engineer*, London, England, March, 1935.
[46] See p. 1358.

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DISCUSSIONS

ECONOMIC DIAMETER OF STEEL PENSTOCKS

Discussion

BY ROBERT W. ANGUS, ESQ.

ROBERT W. ANGUS,[43] ESQ. (by letter).[43a]—The method of determining the economic diameter of penstocks has been presented in an interesting and valuable manner in this paper, and the authors have expressed the results in convenient formulas.

In the investigation the head, H, is defined as "the average head, on the penstock, including water-hammer effect, in feet", but the water-hammer itself may have an important influence on the size of the pipe, and the pressure rise due to this effect may be so large as to make Equation (6) inaccurate. A few years ago the writer was making some studies with a model pipe line having a glass surge tank. The top of this tank was open, was above the forebay level, and was tested and found to be amply strong for pressures produced by the static head in it. On several occasions, however, the glass surge tank was broken by a quick closure of the turbine gate, although no water came over the top of the tank, and a carefully made indicator showed pressures at its base much in excess of those due to the height of the water column.

Of course, it is well known that under favorable conditions, the water-hammer pressure rise, in feet, may be about 100 times the velocity dissipated, and also that these bad conditions may readily occur in a long penstock or pipe. For example, if a pipe is 6 000 ft long, the foregoing pressure rise will occur for valve closure requiring a period of 4 sec, or less, a length of time easily within the operation range of a governor; and if the pipe is under a low head of perhaps 100 ft and has its velocity of, say, 4 ft per sec dissipated, the pressure from the resulting water-hammer will be five times the normal pressure.

NOTE.—The paper by the late Charles Voetsch, M. Am. Soc. C. E., and M. H. Fresen, Assoc. M. Am. Soc. C. E., was published in November, 1936, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: March, 1937, by Messrs. R. A. Monroe, William E. Rudolph, and Peter Bier; April, 1937, by Adolpho Santos, Jr., Assoc. M. Am. Soc. C. E.; May, 1937, by Messrs. Joseph D. Lewin, F. Knapp, and Ralph W. Powell; and June, 1937, by H. K. Barrows, M. Am. Soc. C. E.

[43] Prof. of Mech. Eng., Univ. of Toronto, Toronto, Ont., Canada.

[43a] Received by the Secretary May 3, 1937.

Recently, a case came to the writer's attention, in which two pipes ran parallel to each other from the side of a reservoir and were joined at their lower ends by a **U**-bend. Each pipe had a valve near the reservoir and the valves were under a static head of 66 ft, one valve being open and the other closed at the time. Each pipe also had a turbine connected near its lower end, one of which was shut down, and the other running. The operating turbine was then stopped by a quick, not "sudden", closure of the gates (that is, the closing time exceeded the total time of the pressure wave), but the resulting pressure rise was sufficient to break the closed valve that was near the reservoir. An examination suggested that the resulting pressure was more than twice the static head of 66 ft.

It would appear desirable and necessary, therefore, to include the water-hammer effect in a much more exact manner than by merely taking its average value. In the Symposium on Water Hammer sponsored by the Society, among others,[24] A. W. K. Billings, M. Am. Soc. C. E., has shown that the study of water-hammer modifies the design of the pipe line greatly. In view of the fact that the pressure rise due to this cause may be so easily arrived at graphically,[44] even in fairly complicated systems, it would seem that it should be included in any economic study, although the mathematical expressions for it would be difficult to include in ordinary equations.

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[24] "High-Head Penstock Design", by Messrs. Billings, Dodkin, Knapp, and Santos, Symposium on Water-Hammer, A. S. M. E., Hydraulic Div., and Am. Soc. C. E., Power Div., 1933, pp. 29 *et seq.*

[44] "Simple Graphical Solution for Pressure Rise in Pipes and Pump Discharge Lines", by Robert W. Angus, *Journal*, Eng. Inst. of Canada, February, 1935; also, "Water-Hammer in Pipes, Including Those Supplied by Centrifugal Pumps; Graphical Treatment", by Robert W. Angus, *Proceedings*, Inst. of Mech. Engrs., 1937.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

RECLAMATION AS AN AID TO INDUSTRIAL AND AGRICULTURAL BALANCE

Discussion

BY CHARLES P. WILLIAMS, M. AM. SOC. C. E.

CHARLES P. WILLIAMS,[24] M. AM. SOC. C. E. (by letter).[24a]—A clear and convincing argument regarding what may be accomplished by the co-ordination of agriculture and industrial development, is contained in this paper. In the development of a large irrigation project the economic phases constitute a difficult and paramount problem. Of the irrigation projects built by the U. S. Bureau of Reclamation none can be considered, in itself, a commercially feasible investment. A project that cannot return the construction cost, without interest, within twenty-five years or more, cannot be considered commercially feasible. By no means does it follow that the investments were not justified. Incidental to the agricultural project, towns developed within the irrigated areas, with small mercantile stores of all kinds. There were established sugar factories, alfalfa meal mills, flour mills, creameries, cheese factories, canning factories, packing houses, and other small industries of various kinds. The benefits of the project were not confined to the irrigated area. A market was created for materials and supplies manufactured outside the project. The prosperity of such projects undoubtedly would be greatly increased by the introduction of other industries for supplying not only existing local needs, but also the increased demand created by the increased population.

In the paper, attention is called to the tendency toward the decentralization of industry. Such decentralization has been due, no doubt, not only to the greater economy of relatively small production units, to which the authors call attention, but also to the very high value of land in, or immediately adjacent to, large cities, and the consequent high value of plant sites, high taxes, and necessarily higher wages, due to the increased cost of living, for employees. Such decentralization, of course, has been on the

NOTE.—The paper by Ernest P. Goodrich and Calvin V. Davis, Members, Am. Soc. C. E., was published in November, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: March, 1937, by Joseph Jacobs, M. E. McIver, and C. S. Jarvis; and April, 1937, by Messrs. John P. Ferris and L. C. Gray.

[24] Cons. Engr., Comisión Nacional de Irrigación, City of Mexico, D. F., Mexico.

[24a] Received by the Secretary June 1, 1937.

initiative of the plant owners and management. Incidentally, labor has benefited by a lowered cost of living and more agreeable and healthful living conditions.

The marked increase in unemployment during the relatively prosperous years 1923 to 1929, inclusive, is discussed by the authors, as well as the percentage of decrease in labor forces and the concurrent percentage of increase in production, in each of nine important industries, during the period, 1923 to 1927, inclusive. Various examples have been cited of the remarkable increase in efficiency in various industries by the introduction of machines and more efficient methods, thus reducing the labor necessary, and adding greatly to the army of the unemployed.

The authors are of the opinion that the plan proposed, if adopted generally, would be very effective in reducing unemployment. The writer doubts the validity of this conclusion unless other measures, not mentioned by the authors, are taken. The economic structure of the United States is a very intricate and complicated system, and a complete and comprehensive discussion of the causes of depressions and recoveries is obviously far beyond the scope of this discussion. However, there are certain basic principles affecting the system which, to the writer, appear to be evident.

Undoubtedly, there are many factors which contribute to unemployment. One very important factor, if not the most important, is the introduction of machines and more efficient methods, thus reducing greatly the number of men necessary for the accomplishment of any industrial work. However, no sane man would advocate the discard of machines, or the return to less efficient methods. Even greater efficiency is desirable wherever possible. The problem is to determine how efficiency can be retained, and even increased, and, at the same time, how to eliminate unemployment, or at least, to reduce it greatly.

Assume a country in which there is such industrial and vocational balance that all men capable of working are employed and that all are receiving such just compensation as enables each to satisfy his needs of goods produced and services rendered. Now, let machines and more efficient methods be introduced to such extent that the total number of man-hours of work, in the country as a whole, necessary for the production of those things which before had been produced, and the rendering of the services before rendered, are reduced, say, 10 per cent. If then the industrial and vocational balance is to be maintained, one of the following two plans, or a combination of the two, must be adopted:

(*A*) The total number of hours of work of those engaged in production or service may be reduced 10%, each individual receiving such compensation as will enable him to continue to satisfy his needs of goods produced and services rendered; or,

(*B*) The number of hours of work per individual and the wages or salaries per individual, in the industries and vocations in which machines and more efficient methods have been introduced, may be retained as before, and the number employed may be reduced to that necessary for the same

amount of production and service as before. Those not retained in these industries and vocations may be employed in new industries or services, with appropriate hours of labor and appropriate compensation, or in old industries or services in which the forces employed therein are insufficient to supply the demand for the products or services.

Under Plan (*A*), the cost of production and services, prices, ability to purchase, and profits will remain the same as before. The only advantage will be that: (1) All are employed as before; (2) every one will be able to enjoy the same comforts as before; and (3) the workers will have more leisure for recreation, mental improvement, social activities, and similar avocations.

Under Plan (*B*), each of the workers retained in the former industries and vocations will receive the same wage or salary as before. However, the cost of production and service will be reduced, prices can be reduced, and the worker can buy the products and services before purchased for less than the sum originally expended therefor, leaving a part of his earnings for the purchase of goods produced in the new industries and of the new services introduced. Likewise, those employed in the new industries and vocations will be able to purchase their proportionate parts of the goods originally produced and the services originally rendered, and also the products of the new industries and the services of those in the new vocations.

Plan (*B*) evidently is preferable to Plan (*A*). Under Plan (*B*), a higher standard of living will result. Owners of industries may receive the same profits as before, but at a higher percentage, or they may receive even greater profits if not all the decreased cost of production is applied to a decrease in prices.

Due to the very great reduction in labor needed in industry in recent years, it probably would not be possible to provide sufficient work in new industries and new vocations to reduce unemployment greatly. Such great reduction could probably be accomplished only by the combination of Plans (*A*) and (*B*).

In the plan of combined agricultural and industrial development, suggested by the authors, it is proposed that the workers be employed part-time in the industrial plants and part-time in agriculture. Agriculture is not a new industry; neither is agricultural production insufficient to supply the demand for agricultural products. The decrease in the hours of labor per man in industry would increase the number of men employed in industry, but if the reduction in hours of labor per man in industry is applied to agricultural production, to the same extent would farm labor by those engaged primarily in agricultural pursuits be reduced.

The writer does not wish to be understood as being of the opinion that the Madera District would not be benefited by the plan proposed by the authors. He believes that it would be. More particularly would the industrial workers be benefited by being occupied part-time in healthful, outdoor work, as well as by obtaining increased compensation. As a means of reducing unemployment, however, he believes that it would have little, if any, value.

The writer is of the opinion that, although the co-operative plan of gardening might result in greater efficiency, the allotting of individual tracts to workers would be more beneficial for the reasons stated by the authors. In this case, it is believed that efficiency is of less importance than the psychological effect. It would be desirable to have a demonstration farm where the men could work for a time until they become acquainted with effective agricultural methods. At first, this farm could be of large area (embracing the entire 25 000 acres, in the case of the Madera District). Individual allotments could then be made therefrom as the men become competent to conduct their own operations efficiently. The stabilizing influence, to which the authors refer, would be much greater if each man had a proprietary interest in his own individual tract than if his work were on a co-operative basis. As far as conditions would permit, each could live on his tract and could establish a permanent home on it. It is true that a centralized plan would be necessary, but it need not be co-operative, except for single men and those lacking agricultural training.

The authors propose that the managing, and presumably the financing of the industrial gardens be by the several industrial companies. The writer believes this to be inadvisable. Some industrial owners may be philanthropic and altruistic, but, in general, their chief aim naturally is to secure a maximum profit. Company farms, like company stores, and large tenant plantations might become a force for economic slavery. The agricultural unit, including houses for workers, could easily be made a State, or National Government, self-liquidating project, financed in a manner similar to the operation of the Federal Housing Administration, the manager, agriculturist, and all assistants being State employees.

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DISCUSSIONS

CONSTRUCTION AND TESTING OF HYDRAULIC MODELS MUSKINGUM WATER-SHED PROJECT

Discussion

BY RALPH W. POWELL, M. AM. SOC. C. E.

RALPH W. POWELL,[10] M. AM. SOC. C. E. (by letter).[10a]—Hydraulic model testing has become well recognized as an essential part of hydraulic design, and, therefore, the authors have placed all hydraulic engineers in their debt by sharing valuable information as to methods and costs. During the period covered by the tests, the writer was serving as Hydraulic Engineer for the Muskingum Watershed Conservancy District, and had the pleasure of observing several of the tests in progress, and of making a fairly thorough study of the complete reports[10b] which were prepared. This contact gave the writer a high regard for what was accomplished under very definite limitations.

As far as the writer is aware, no equal amount of model testing has ever been done in one laboratory in an equal length of time. Thirteen models were constructed and tested in about ten months. This was possible only by working the models on three 8-hr shifts, and meant 13 or 14-hr days for the Supervisory Staff. Although the authors are to be congratulated on the work done under such conditions, they would probably be the first to admit that the results might have been better if more time could have been taken. Engineers who wish model tests comparable to those described should, if possible, allow at least three months for each model. If the facilities of the laboratory are sufficient, several models may be tested simultaneously, but in this case the number of the experienced personnel must be increased accordingly.

NOTE.—The paper by George E. Barnes, M. Am. Soc. C. E., and J. G. Jobes, Jun. Am. Soc. C. E., was published in December, 1936, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows, May, 1937, by Messrs. G. W. Howard, F. W. Edwards, and T. T. Knappen.

[10] Assoc. Prof. of Mechanics, Ohio State Univ., Columbus, Ohio.

[10a] Received by the Secretary July 26, 1937.

[10b] It should be noted that copies of these reports can be secured at the cost of reproduction. Inquiries concerning prices should be addressed to Professor Barnes. Reports on five of the dams are also on file at Engineering Societies Library, New York, N. Y. (See *Civil Engineering*, August, 1937, p. 586.)

The problem of piezometer readings has already been discussed. In view of the uncertainty of many of the readings the writer wonders whether the hook-gages were not an unnecessary refinement and whether direct reading manometers arranged in "gangs" on boards would not have been sufficient. This arrangement has the advantage that the prototype elevations can be marked plainly on the board, and a photograph will give a permanent record of all the readings at the same instant. If any tube is much "out of line", it will be noticed at once.

The value of 0.0091 for Manning's n in the Mohawk model is entirely possible, but the writer does not feel that much weight can be given to such measurements. Prandtl and Tietjens[11] state that in the case of turbulent flow, the kinetic energy head increases from $\frac{V^2}{2g}$ to $1.09 \frac{V^2}{2g}$ in the first 40 diameters of the length of the pipe. In a complicated structure like the outlet works of the Mohawk Dam, it is somewhat of a question what point to consider as the entrance, but using the gate slots, the points at which the pressure was measured were approximately 11 and 21 tunnel diameters down stream from the entrance, respectively. How much of the $0.09 \frac{V^2}{2g}$ increase in kinetic energy head took place between these two sections is difficult to say, but it may easily have been as much as $0.02 \frac{V^2}{2g}$. This would be of the order of 10% or more of the entire pressure drop observed. This fact, coupled with the admitted uncertainties in the piezometer readings, makes the figure 0.0091 of doubtful value. This is entirely apart from the doubt which grows constantly in the writer's mind as to whether Manning's formula should be used in model work at all. A better means of expressing fluid friction in conduits is badly needed.

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[11] "Applied Hydro- and Aero-Mechanics", by Prandtl and Tietjens, McGraw Hill Book Co., 1934, pp. 49–51.

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DISCUSSIONS

GRAPHICAL DISTRIBUTION OF VERTICAL PRESSURE BENEATH FOUNDATIONS

Discussion

By A. A. Eremin, Assoc. M. Am. Soc. C. E.

A. A. Eremin,[38] Assoc. M. Am. Soc. C. E. (by letter).[38a]—An interesting method of computing foundation stresses by means of the Boussinesq equation, is contained in this paper. Mr. Burmister has applied his method to footings with uniform pressures. However, Fig. 3 may also be used in computing foundation stresses beneath footings with non-uniform pressure distribution. The areas of such footings should be expressed in terms of a transformed area. For instance, assume that the pressures in terms of Footing No. 2 (Fig. 16), are 3, 4, 6, and 5 tons per sq ft at Corners *a*, *b*, *c*,

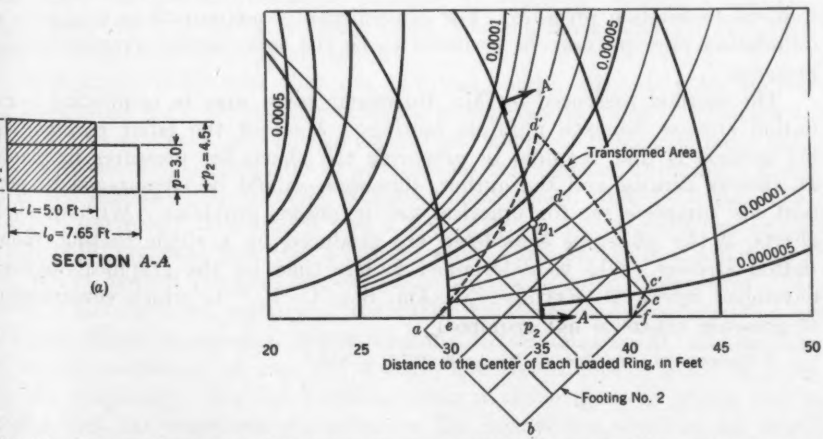

Fig. 16.—Transformed Area of Footing No. 2. Pressure Beneath Footing, *p*, in Tons per Square Foot; Pressure Chart and Footing No. 2 Same as That Shown on Fig. 3.

Note.—The paper by Donald M. Burmister, Assoc. M. Am. Soc. C. E., was published in January, 1937, *Proceedings*. Discussion on this paper has been published in *Proceedings*, as follows: May, 1937, by Messrs. William B. Kimball, I. M. Nelidov, George Paaswell, and Jacob Feld; and June, 1937, by Messrs. Nathan M. Newmark, A. E. Cummings, and D. P. Krynine.

[38] Assoc. Bridge Designing Engr., State Highways, Bridge Dept., Sacramento, Calif.

[38a] Received by the Secretary June 19, 1937.

and d, respectively. In Section A–A along an arc with a radius of 35 ft from Point A, the pressures vary from $p_1 = 4.66$ tons per sq ft to $p_2 = 4.50$ tons per sq ft (see Fig. 16(a)). By changing the length, l, the area of the pressure diagram may be expressed as an equivalent area of uniform pressure. The length, l_0, of the footing in Section A–A, Fig. 16(a), with a uniform pressure of $p = 3$ tons per sq ft, is,

$$l_0 = \frac{(p_1 + p_2)\,l}{2p} \quad (21)$$

or, $l_0 = \dfrac{(4.66 + 4.50)\,5}{2 \times 3.0} = 7.65$ ft.

The width of the transformed area may be constructed along the arcs at various distances from Point A. The computed area of the footing, with a uniform pressure of $p = 3$ tons per sq ft, is shown in Fig. 16 by dotted lines, e, d', c', and f. Area $abfe$ (Fig. 16) is revolved 180° about Line ef and is also converted or transformed into an equivalent uniform pressure. Foundation stresses beneath this transformed area may be computed as shown by the author in Fig. 3.

If the pressure curves under the footings are irregular, the area of the pressure diagram along the arcs may be determined by planimeter measurements, or by approximate summation. The width of the transformed footing along an arc, l_0, will be,

$$l_0 = \frac{A_p}{p} \quad (22)$$

in which A_p is the area of the pressure diagram, and p is the assumed uniform foundation pressure. For convenience of computations the uniform foundation pressure may be assumed to be the same as the average footing pressure.

The method presented by Mr. Burmister saves time in computing foundation stresses beneath multiple footings. Most of the labor involved in his method is that required in preparing the charts for pressures in planes at various depths, and the author, therefore, should be encouraged to present his complete set for practical use in design problems. Without such charts, if the pressures considered are produced by a single footing, foundation stresses could be determined in less time by the graphical method developed by D. P. Krynine, M. Am. Soc. C. E.,[39] in which construction of pressure charts is not required.

[39] *Proceedings*, Am. Soc. C. E., April, 1937, p. 669.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

STRUCTURAL ANALYSIS BASED UPON PRINCIPLES PERTAINING TO UNLOADED MODELS

Discussion

BY MESSRS. F. B. FARQUHARSON, AND A. A. EREMIN

F. B. FARQUHARSON,[26] ASSOC. M. AM. SOC. C. E. (by letter).[26a]—The writer views this paper as a contribution to the field of analysis of structures by models and from this vantage point suggests that it provides a real contribution in a field that receives altogether too little attention from the practicing structural engineer. It has been the writer's experience that the influence line in general takes on no real and lasting significance until experienced and handled in the form of a model. After working with several model methods for a number of years, the re-thinking necessary in reading this paper has served a valuable function in bringing Maxwell's theorem, slope-deflection principles, influence lines, and the model and its prototype into a more harmonious relationship.

It is undoubtedly true that the fact that models may only be studied in connection with an existing tangible model, has led many engineers to neglect this approach in favor of one of the many methods requiring only pencil, paper, and a slide-rule as adjuncts to fundamental thinking. Thus, it is probable that the majority of efforts along model lines have been confined to university laboratories and a few of the larger engineering offices.

It is unfortunate that this condition exists since it does not follow that the model method requires the use of an expensive set of equipment. Indeed, it has been the practice of the writer to accompany the discussion of influence lines by practical demonstrations on the blackboard, making use of a long wooden spline and half a dozen small steel pins inserted in holes in the blackboard. In this manner, using a sharp piece of chalk and a meter stick for measurement, influence line results are obtained on fairly complex structures which check theoretical computations to within 15 per cent. Of course, the general usefulness of the model method is much ex-

NOTE.—The paper by Otto Gottschalk, Esq., was published in January, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: March, 1937, by Messrs. L. J. Mensch, and Frederick Shapiro; and May, 1937, by Messrs. James R. Griffith, and Camillo Weiss.

[26] Assoc. Prof. of Civ. Eng., Dept. of Civ. Eng., Univ. of Washington, Seattle, Wash.

[26a] Received by the Secretary, July 23, 1937.

tended by the availability of accurate and more elaborate equipment offered commercially for this purpose.

It has been the writer's experience that much of the indifference to the use of models arises through engineers who have never tried the method. It is suggested that even so complex a problem as that illustrated in Fig. 1 of the paper might be solved with an accuracy within 10% or 15%, through the use of a few carefully selected wooden splines, a dab of glue, and a few brads. The model can be constructed and mounted in an hour. A minute or two will suffice to trace off the influence line from which the moment at Point E is observed directly for any single concentrated load and obtained with ease for any load distribution whatever. To be sure, a check solution yielding the exact theoretical influence line may occupy several evenings, but the result is likely to be an enthusiasm for the model as an adjunct to structural analysis.

A. A. Eremin,[27] Assoc. M. Am. Soc. C. E. (by letter).[27a]—An interesting method of computing bending moments and shear stresses in rigid frames, based on the Mohr principle, which has been cited by the writer elsewhere,[28] is developed by Mr. Gottschalk. His method has merit, but, it is regrettable that he failed to extend it to the computation of stresses in rigid frames with variable sections. The advantage of rigid frames and continuous beams with variable sections is evident and it is a serious limitation of the method that it can be applied only to structures with constant sections.

The computations may be divided into two steps: (1) Computing the stiffness factors, S, at each end of members in the frame; and (2) computing the factors, f. The stiffening factors express the elastic properties of a frame and do not vary with loads on the frame. The factors, f, computed in Step (2), however, vary with the manner of loading and the location of a section in the frame at which analysis of stresses is desired. The final bending moments and shear stresses in the frame are determined by the algebraic expressions involving the Factors f and the loading. The method may be extended to determine stresses in rigid frames with variable sections. Computation of the stiffening factors, S, and loading factors, f, for variable sections may be simplified by preparing tables of constants for the elastic properties of members in frames. However, the final algebraic expression for bending moment and shear stress in members with variable sections, involving the factors, f, will be lengthy and difficult of solution for practical purposes.

In rigid frames more than one story in height the value of S, cannot be obtained by direct computations. It must be assumed, and true values are determined by repeated trial computations. Therefore, the labor required for computing the values of S varies with the experience acquired in deciding on an initial trial value.

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[27] Associate Bridge Designing Engr., Bridge Dept., State Highways, Sacramento, Calif.

[27a] Received by the Secretary July 23, 1937.

[28] *Transactions*, Am. Soc. C. E., Vol. 96 (1932), p. 21.

A direct computation of shear stresses by Equations (11) and (12) can scarcely be considered a saving of time. In rigid frames, shear stresses at ends of members are generally determined by the formula:

$$V = V_0 + \frac{M_1 - M_2}{1} \ldots\ldots\ldots\ldots\ldots\ldots\ldots (68)$$

in which V_0 = shear stress at one end of a member assuming both ends freely supported; M_1 and M_2 = the restraining bending moments at the ends of a member; and l = span length of the member. Evidently, the time and effort required for computing Factor f and the shear stresses is not less than that required for computing the restraining bending moments, M_2 and M_1, and shear stress, V_0, in Equation (68).

The author's method is convenient in the construction of influence lines for bending moments and shear stresses in rigid frames. Likewise, it may be used conveniently in the analysis of stresses in rigid frames and continuous beams that have a single statically indeterminate reaction. However, in computing bending moments and shear stresses for each member of a multiple-span rigid frame the method involves lengthy computations and cannot be considered as a time saver.

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DISCUSSIONS

RAINFALL INTENSITIES AND FREQUENCIES

Discussion

By C. H. Eiffert, M. Am. Soc. C. E.

C. H. Eiffert,[37] M. Am. Soc. C. E. (by letter).[37a]—The writer is very much interested in this paper and is pleased to note that the authors apply the method of combining years of record of a number of stations to be used in the determination of frequencies. This method was used extensively in one of the Technical Reports of the Miami Conservancy District, "Storm Rainfall of the Eastern United States", published in 1917. This report was revised and reprinted in 1936. Elsewhere,[38] the writer has attempted to emphasize some of the possibilities of this method for increasing the usefulness of precipitation records of comparatively short duration. The Miami Conservancy District studies were of great storms lasting from 1 day to 5 days and were made for flood-control purposes, whereas the investigations described by the authors were made in connection with sewer design and refer to excessive precipitation lasting from 5 min to 2 hr. However, the principle of combining the years of record is the same.

It is perhaps pertinent to state that judgment and caution must be used in the application of this method. The area from which the records are combined must have uniform rainfall characteristics; otherwise, the results will not be correct for different points within the area. For instance, an area in which there are great differences in altitude would probably also have great differences in precipitation both as to type and quantity. The years of record for stations in such an area could not be combined satisfactorily.

The records for the individual stations should be long enough to cover a fairly complete rainfall cycle. Such cycles are indefinite at best; yet it is well known that periods of wet and dry years do occur. Therefore, any record so short that it covers a wet or a dry period only would not be suitable for use in this connection.

Note.—The paper by A. J. Schafmayer, M. Am. Soc. C. E., and the late B. E. Grant, Esq., was published in February, 1937, *Proceedings*. Discussion on this paper has been published in *Proceedings*, as follows: April, 1937, by Messrs. Victor L. Cochrane, and L. K. Sherman; and June, 1937, by Messrs. J. O. Jones, Charles W. Sherman, Glen N. Cox, Garrett B. Drummond, Eugene L. Grant, Adolph F. Meyer, and Clinton L. Bogart.

[37] Chf. Engr. and Gen. Mgr., The Miami Conservancy Dist., Dayton, Ohio.

[37a] Received by the Secretary July 2, 1937.

[38] *Engineering News-Record*, December 31, 1936.

In the aforementioned work of the Miami Conservancy District, the eastern half of the United States was divided into 2° quadrangles. The records of the stations in each of these quadrangles were added to obtain the total years of record for the quadrangle. Records of less than 5-yr duration were discarded. The size of the quadrangles used is somewhat arbitrary. It should probably be changed in order to obtain the most satisfactory results for certain localities. In the study of the entire eastern half of the United States this refinement could not be attempted on account of the work involved.

There seems to be no good reason for combining the records of stations as far apart and differing as much in climatic character as Boston, Mass., Yankton, S. Dak., and Knoxville, Tenn. The combination of records from stations in the Chicago (Ill.) area should be quite satisfactory. It is true that the records of a number of the stations are much shorter than is desirable so that only part of a cycle is covered and this fact should be borne in mind when the data are being used.[38a] Even if the period is short it is probably the best possible way of using these records. It will be necessary to revise the curves from time to time as records accumulate.

A very valuable publication on "Rainfall Intensity-Frequency Data", by the late David L. Yarnell, M. Am. Soc. C. E., was issued by the U. S. Department of Agriculture in August, 1935. This record covers periods from 5 min to 24 hr. The values taken from the authors' curves for Chicago agree very closely with those for the same location taken from Mr. Yarnell's maps.

The writer has seized every opportunity in the past to emphasize the necessity for obtaining more complete records of rainfall and run-off, and cannot refrain from doing so again in this connection. The U. S. Weather Bureau needs many more recording rain-gages. The records should be made continuous and should not be subject to termination on account of lack of appropriations as they are at the present time. Fragmentary records lose a great deal of their value for the aforementioned reason, namely that they cover only parts of cycles. Engineers should emphasize the importance of such records whenever possible because legislators, as a rule, do not realize the necessity for them.

[38a] Correction for *Transactions:* Pages 234–235 (February, 1937, *Proceedings*), Table 4, Column (11) read "Dodge City, Kans.", instead of "Dodge City, Iowa"; page 1130, Line 28 (June, 1937, *Proceedings*), change "4 yr" to read "40 yr".

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DISCUSSIONS

FLOW CHARACTERISTICS IN ELBOW DRAFT-TUBES

Discussion

BY ELLERY R. FOSDICK, ESQ.

ELLERY R. FOSDICK,[27] ESQ. (by letter).[27a]—The improvements which heretofore have been made in the science of hydraulics as related to power production, have been almost entirely in the field of dynamics and have usually been confined to such equipment as turbines, needle-valves, the velocity-recovering ability of draft-tubes, and intakes to flow lines. Practically no attempts have seemingly been made to improve the design and operating characteristics of such important structures as bends, elbows, and wye-branches. The excellent work presented in this paper by Professor Mockmore, therefore, is a valuable addition to the information that is available on the subject of losses in conduit bends.

The losses that result from bends and wye branches in conduits are frequently quite large and seem to justify more consideration than is usually given them. They are of sufficient magnitude in some installations to warrant the expenditure of additional capital for their reduction.

Draft-tubes are normally subjected to very low heads or to a partial vacuum whereas conduit bends and wye branches operate under higher heads. It follows, therefore, that the type of design which is most suitable for a draft-tube may not be practicable for use in conduit bends and wye branches since for this type of structure it must possess mechanical stability under high heads and, at the same time, be justifiable economically.

The improvements that have been made to hydraulic structures, other than bends and wye branches, have reduced the losses occurring in them to such a low point that no material improvements can be expected in the future and, therefore, the only changes which it is likely will be made are refinements that may result in very minor increases in efficiency. On the other hand, the designs of bends and wye branches have not been changed,

NOTE.—The paper by C. A. Mockmore, M. Am. Soc. C. E., was published in February, 1937, *Proceedings*. Discussion on the paper has appeared in *Proceedings*, as follows: April, 1937, by F. T. Mavis, M. Am. Soc. C. E.; May, 1937, by Jerome Fee, Assoc. M. Am. Soc. C. E.; and June, 1937, by Messrs. R. E. B. Sharp, and L. F. Harza.

[27] Elec. Engr., Spokane, Wash.

[27a] Received by the Secretary June 14, 1937.

for the most part, since the inception of conduits, having a circular cross-section, and the losses in these structures are sufficiently large in some instances to justify the expenditure of additional sums of money, in order to improve the designs toward the end of increasing their efficiency and improving their operating characteristics.

General.—In view of the foregoing facts, the writer has conducted an investigation during the past few years similar to that made by Professor Mockmore, to determine the character of the losses that occur in bends and wye branches, whereas the different types of bends investigated by Professor Mockmore were principally suited for use under low heads as contrasted with the types of structure that were studied by the writer for use under high heads.

The passage of a fluid around a bend in a conduit results in a double spiral eddy in which the diversion loss occurs. This eddy is produced as a result of the distortion to the normal distribution of velocity and an abnormal pressure created by the centrifugal thrust of the fluid. These phenomena have been studied by several investigators,[28] and the results of their work have been previously made available to the Engineering Profession through the medium of various technical publications. Consequently, no further discussion of the subject will be given herein.

It has been known to hydraulic engineers for some time that the losses resulting from a bend may be decreased materially through the use of a conduit flattened in a plane at right angles to the plane or the bend. Such a structure, however, is suitable only for very low heads and could not be made physically stable for high heads without excessive cost.

In view of these facts, the selection of a design for reducing the losses in conduit bends would be more or less limited to a cross-sectional shape that was circular, or some part of a circle, so that it would tend to maintain its normal contour under high pressures. After much investigation of this problem before the beginning of these tests, it was finally decided to study a bend with a semi-circular cross-section with the shortened diameter lying in the plane of the bend. Such a conduit might be easily constructed for use under high pressures from a circular pipe by merely installing a diametrical partition wall with relatively small vent holes in it, which would transmit the fluid pressure to the side of the partition wall away from the moving column of water. In this manner the hydrostatic pressure of the fluid would be exerted upon a physically stable structure with a circular cross-section, while the dynamic flow would take place through a conduit having a semi-circular cross-section and the partition wall would only have to withstand the forces produced by the dynamic flow of the fluid passing around the bend.

The tests that are described in this discussion were conducted in the Hydraulics Laboratory at Washington State College, Pullman, Wash., using

[28] "Flow of Water Around Bends in Pipe", by the late David L. Yarnell and Floyd A. Nagler, Members, Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 100 (1935) p. 1018; "Modern Conceptions of the Mechanics of Fluid Turbulence", by Hunter Rouse, Assoc. M. Am. Soc. C. E., *Proceedings*, Am. Soc. C. E., January, 1936, p. 21; "Loss in 90-Degree Pipe Bends of Constant Circular Cross-Section", by Albert Hofman, *Transactions*, Munich Hydr. Inst., *Bulletin 3*.

a standard 6-in. pipe for the supply and discharge lines from a 90° test bend. A schematic arrangement of the flow line which was used for these tests is shown in Fig. 20. It will be observed that the supply line was a

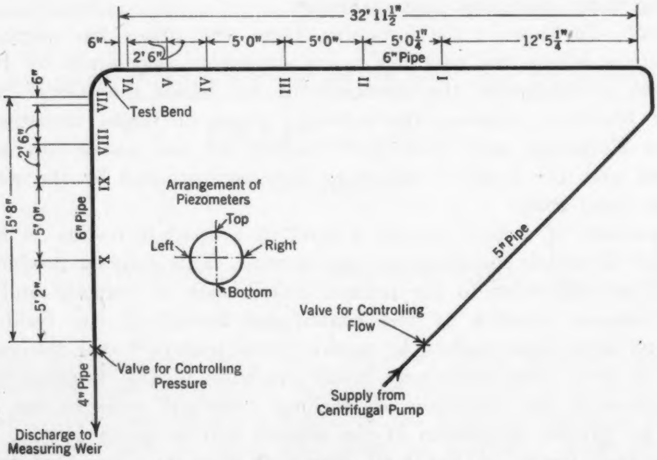

FIG. 20.—DIVERSION LOSSES IN PIPE BENDS; ARRANGEMENT OF FLOW LINE USED FOR TESTS.

straight section of 6-in. pipe practically 33 ft long, whereas the discharge pipe was a straight section of 6-in. pipe approximately 16 ft long. The 90° test bends were inserted between these two sections of pipe. The water supply was obtained from a centrifugal pump, and the flow was measured over a triangular weir shown in Fig. 21, which had previously been cali-

FIG. 21.—CONCRETE FLUME AND TRIANGULAR MEASURING WEIR.

brated within very close limits. The flow was regulated by means of a valve on the discharge side of the pump and the pressure on the flow line and test bend was regulated by means of a valve in the discharge pipe.

Ten measuring sections were laid out along the 6-in. pipe, as shown in Fig. 20. Four piezometers were placed at each of these sections, spaced 90° apart, with two of them lying in the plane of the bend and two in the plane at right angles to the plane of the bend. Each of these piezometers consisted of a $\frac{1}{16}$-in. hole which had been drilled radially into the pipe and reamed with a specially built reamer so that the broken bits of metal were removed from the inside of the pipe at the edge of the hole and also so that the inside edge of the hole was slightly rounded. Figs. 22 and 23 show the supply line and a test bend in place in the flow line and the discharge line.

FIG. 22.—SUPPLY LINE AND TEST BEND.

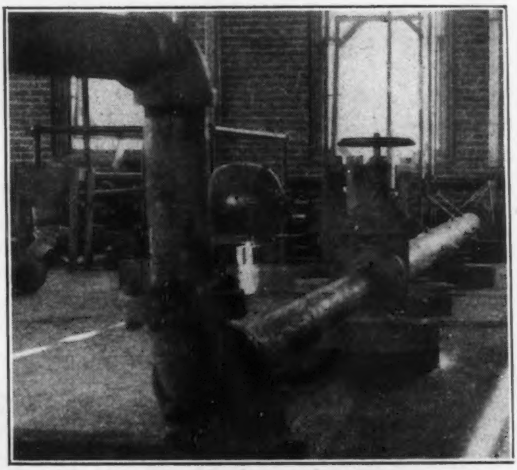

FIG. 23.—DISCHARGE LINE AND CONTROL VALVE.

Each of the piezometers was connected to a vertical, glass tube manometer. Throughout the tests of the first two bends, the oscillations of the water column in the manometers were damped by pinching the rubber tubes that connected the manometers to the piezometers. During the tests on the third bend a 6-in. length of capillary tube was placed in series between each manometer and the piezometer for the purpose of damping these oscillations. This latter arrangement was found to be somewhat more satisfactory than the one originally used.

Three types of 90° bends were tested; the first had a semi-circular cross-section with the same area as a standard 6-in. pipe with the flat side of the semi-circular cross-section at right angles to the plane of the bend and on the side of the pipe away from the center of curvature. This bend was connected to the pipe having a circular cross-section by means of transition sections. After some preliminary tests had been run on this particular bend another one was constructed, in which the transition and the bend were cast integral in one piece. This bend is shown in Fig. 24.

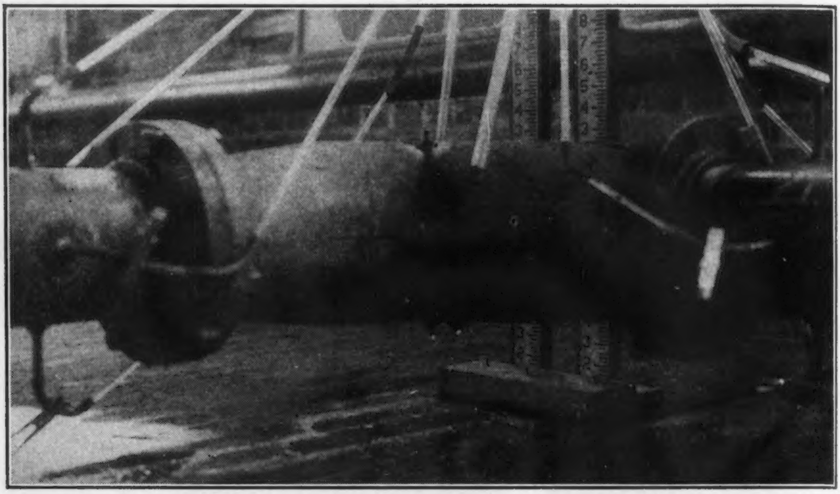

FIG. 24.—FRONT VIEW OF IMPROVED SPECIAL ELBOW NO. 1.

The next set of tests was run on a standard 6-in. flanged, short-radius elbow. The third set of tests was run when using a semi-circular bend with a cross-sectional shape similar to that in the first set, except that the flattened surface was located on the side of the bend nearest the center of curvature, as shown in Figs. 25 and 26. All the special bends that were

FIG. 25.—END VIEW OF IMPROVED SPECIAL ELBOW NO. 2.

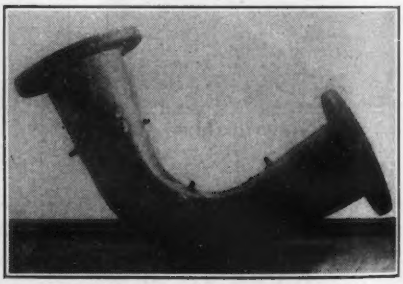

FIG. 26.—SIDE VIEW OF IMPROVED SPECIAL ELBOW NO. 2.

tested had the same radius of curvature on the face of the pipe farthest from the center of curvature as the standard bend. A total of 10 564 observations was made of the losses and corresponding flows for the first two of these bends, and 4 496 observations were made of the third one. The results of these observations have been computed and analyzed graphically. A summary of the information which has been obtained follows.

Method of Testing.—Observations of pressure heads were made in the manometers connected only to the right-hand piezometers at the various stations along the flow line, and the equation between the water level in

the right-hand and left-hand piezometer at each station was carefully observed throughout the test runs, and recorded at the end of each group of readings. It was found that the equation between the right-hand and left-hand piezometers at any one station remained substantially the same for any given condition of flow, and for this reason it was unnecessary to take readings in both of them.

A group of twenty-five simultaneous readings of the pressure head in the flow line was made for each condition of flow at each station when testing the standard bend and Special Bend No. 1. These readings were made at 10-sec intervals and were found to cover at least one complete cycle of the variations in pressure that were found to be present in the flow line under stable conditions of flow. Consequently, the average of these twenty-five readings for any one flow completed a fairly accurate measure of the normal average pressure head that existed during the particular run.

In making the observations for Special Bend No. 2, the number of consecutive simultaneous readings that was made for any one condition of flow, was reduced to twenty, as this appeared to be adequate for the degree of accuracy desired. The method of reading and the interval of time between readings were the same as for the previous tests.

Due to the limited number of observers it was impossible to read all the piezometers simultaneously and, therefore, it was necessary to take three groups of readings for each test run at each of four adjacent stations. The stations that were read in each group of observations had to overlap in order to determine a continuous hydraulic gradient for the entire flow line; that is, the last, or fourth, station in the first and second groups of readings were the first stations read in the second and third groups of readings.

Observations of the flow were made by taking readings of the head on the weir at 30-sec intervals throughout each test run. The head on the weir was measured with a standard hook-gage and the observations were read to 0.0001 ft. The standard bend was tested under fifteen different conditions of flow with a range of velocity from 2.5 to 7.2 ft per sec. Improved Bend No. 1 was tested with a total of twenty-one different flows, having a range of velocity from 2.4 to 7.1 ft per sec and Improved Bend No. 2 was tested for nine different flows having a range of velocity from 1.7 to 7.5 ft per sec.

Computation of Results.—The diversion loss resulting from the fluid passing around the bend was found by subtracting the total friction loss in the flow line from the total loss in the flow line.

The temperature of the water passing through this flow line during the tests was measured at frequent intervals, and, from this, it was possible to ascertain the kinematic viscosity. With this factor and the mean velocity of the fluid and the diameter of the pipe known, it was possible to compute Reynolds number for the various conditions of flow. This affords a dimensionless parameter for designating the results of these tests so they may be transposed to other hydraulic structures having the same geometric similitude.

It will be observed from Fig. 27, which contains a summary of the information obtained in these tests, that the diversion loss factors for the standard and special bends approach a constant value at about $\mathbf{R} = 250\,000$.

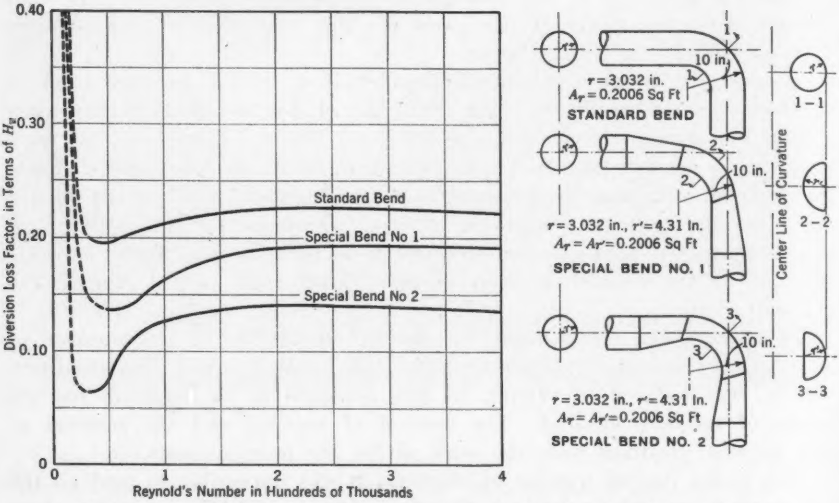

FIG. 27.—COMPARISON OF DIVERSION LOSS FACTORS FOR VARIOUS TYPES OF CAST-IRON FLANGED 90-DEGREE BENDS

This is in accordance with the observations of other investigators and indicates that the loss factors for a Reynolds number of approximately 250 000 may be used with a fair degree of accuracy in computing the losses with a flow having a Reynolds number many times greater.

No observations were made for a Reynolds number much less than 50 000 as this represents a very low mean velocity of flow for a pipe of this size and is a condition that is not ordinarily of importance in the operation of hydraulic structures, therefore, although it may be of theoretical interest, it is not of any practicable importance. It is probable, however, that if these observations had been made, the loss factor curves would have followed approximately the course indicated by the dashed lines on Fig. 27.

Conclusion.—The results of the tests summarized in Fig. 27 indicate clearly that a considerable reduction can be made in the diversion losses that result from the passage of a fluid around a conduit bend by the use of conduits having a semi-circular shape when they are arranged with the flat side at right angles to the plane of the bend. This is in agreement with the conclusions reached by Professor Mockmore and others that the losses resulting from the passage of a fluid around a conduit bend can be reduced by flattening the structure in the plane of the bend. The semi-circular shape of the conduit bends that have been used in these tests create a redistribution of the fluid velocity that minimizes the losses by

reducing the unbalance of pressure resulting from the centrifugal thrust of the fluid column and by decreasing the resistance to flow around the bend on the side nearest the center of curvature. The types of bends tested by the writer were designed to withstand high heads in addition to reducing the losses whereas the bends that were tested by Professor Mockmore were designed for use under low heads. As a result there is not sufficient similarity between the different bends that were tested to permit a comparison of the results.

The loss factors for Special Bend No. 1 with the flat side radially outermost were found to be materially less than the corresponding loss factors for a standard bend of circular cross-section having the same cross-sectional area and radius of curvature. The loss factors for Special Bend No. 2, which had the flat side radially innermost, were found to be much less than the corresponding loss factors for Special Bend No. 1 and very much smaller than the losses for the standard bend.

The construction of a conduit bend having a semi-circular shaped passage for the moving fluid is feasible for operation under high heads with the shell and partition design. In this type of structure a circular pipe having approximately twice the required cross-sectional area, and designed to withstand the fluid pressure, has a relatively thin longitudinal partition wall that is secured in the pipe in a diametrical position in the plane at right angles to the plane of the bend. Relatively small holes in the partition wall transmit the fluid pressure to both halves of the conduit with the result that the only force acting on the partition wall is the relatively small pressure produced by the dynamic action of the fluid.

Acknowledgment.—The facilities of the Hydraulics Laboratory at Washington State College were made available for these tests by H. V. Carpenter, Dean of the Engineering School, and M. K. Snyder, M. Am. Soc. C. E., Head of the Civil Engineering Department. J. G. Woodburn, Assoc. M. Am. Soc. C. E., contributed much of his time and energy in assisting with the work that was done in the Laboratory, and it was largely through his efforts and co-operation that these tests were successfully completed.

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DISCUSSIONS

NATIONAL ASPECTS OF FLOOD CONTROL
A SYMPOSIUM

Discussion

BY MESSRS. H. K. BARROWS, IVAN E. HOUK, AND JOHN E. FIELD

H. K. BARROWS,[55] M. AM. SOC. C. E. (by letter).[55a]—Mr. Wolman's statement of the factual situation in regard to flood protection is in general accord with the views of the writer.

Engineering studies of past destructive floods are generally incomplete, due to the lack in the past of effective agencies to procure and co-ordinate at or about the time of the flood the necessary information to serve as a basis for its study. Fortunately, there are some exceptions, however, as illustrated by the Ohio flood of 1913, where local agencies obtained much valuable data—particularly by the work of the Miami Conservancy District and its excellent set of engineering reports. Even in this case the data of flood damages—particularly those of an indirect nature—are uncertain. During the past decade the U. S. Geological Survey has developed a policy of procuring data of flood run-offs, following up all major floods, and is covering this field admirably.

In general, it is not likely, therefore, that much more specific data of value relating to the earlier destructive floods can be obtained, due to a dearth of definite information from the past.

The lack of detailed and carefully prepared complete programs for flood relief is quite obvious, in most sections of the United States, and little attention has been given as yet to the consideration of other water uses, in co-ordination with flood control.

As Mr. Wolman states, every major flood brings a popular clamor for action which if hastily acted upon, without careful and deliberate planning, is bound to mean wasteful and perhaps ineffective measures.

NOTE.—This Symposium was presented at the Fall Meeting of the Society and at the meeting of the Waterways Division, Pittsburgh, Pa., October 13 and 14, 1936, and published in March, 1937, *Proceedings*. Discussion on this Symposium has appeared in *Proceedings* as follows: June, 1937, by Messrs. F. C. Scobey, Howard T. Critchlow, T. T. Knappen, M. C. Tyler, Gordon R. Williams, Arthur T. Safford, W. G. Hoyt, J. D. Arthur, Jr., John H. Meursinge, H. K. Barrows, E. D. Hendricks, and Edward W. Bush.

[55] Prof., Hydr. Eng., Mass. Inst. Tech.; and Cons. Engr., Boston, Mass.

[55a] Received by the Secretary July 2, 1937.

The inhibitions of the Copeland Act of 1936, which was directed solely at flood-control measures, have had an unfortunate effect in causing almost a complete lack of comprehensive planning of river developments thus far in accord with all the other aspects of river conservation and control. Moreover, in many sections of the country flood-relief benefits are not sufficient to warrant the expenditures required for this purpose alone.

Although the destructive aspects of a river in flood must be reckoned with, the development of its constructive and useful qualities prevailing most of the time should certainly not be overlooked and flood-relief measures are only a part of the general plans that should be made and consistently followed in the full development of the streams of the country.

IVAN E. HOUK,[56] M. AM. SOC. C. E. (by letter).[56a]—Comprehensive treatments of flood-control problems presented in this Symposium show that the disastrous flood occurrences of recent years are gradually causing the development of a long needed, national, flood-prevention movement. Some of the Federal plans outlined by Colonel Covell may be delayed for many years, possibly until future flood calamities provide new impetus. Nevertheless, if only a few are carried to completion at this time, they will result in saving of life and property which can scarcely be evaluated on a purely monetary basis.

Probably one of the most urgent needs in connection with present flood-control work is an adequate forecasting system. Officials of the U. S. Weather Bureau have rendered excellent service on some of the larger rivers where appreciable time intervenes between the occurrence of storm rainfall and the arrival of flood peaks. However, much remains to be accomplished in developing adequate forecasting organizations for small tributary streams in the upper parts of the drainage areas where maximum flood stages occur only a few hours, or only one day or two days, after the most intense precipitation.

The requirements for an ideal river and flood service mentioned by the late Mr. Hayes,[56b] if provided in sufficient number in each river section, should furnish the necessary facilities for forecasting maximum stages during most flood-producing storms. Naturally, they would not be adequate for forecasting cloudburst floods in isolated mountainous drainage courses, where crest stages arrive almost simultaneously with the first run-off (as at Heppner, Ore., on June 14, 1903, when two hundred people were drowned, one-third the town washed away, and approximately $250 000 worth of property destroyed).

Cloudburst floods of the Heppner type occur so suddenly and erratically that their prediction probably never will be practicable. In such cases residents along the lower sections of the streams usually must depend on telephone warnings from up-stream locations. During the Cherry Creek flood of August 3, 1933, caused by cloudbursts south of Denver, Colo.,

[56] Senior Engr., U. S. Bureau of Reclamation, Denver, Colo.

[56a] Received by the Secretary July 6, 1937.

[56b] Mr. Hayes died November 16, 1936.

and the resulting failure of the Castlewood Dam, telephone and radio warnings were so effective that only two lives were lost.[57] Probably the only way to insure warnings of advancing cloudburst floods, if such warnings should ever be considered necessary, would be to install automatic float-gages at up-stream locations, and to connect them electrically with the forecasting office, so that alarms would be sounded when dangerous stages were reached.

Automatic recording river and rainfall gages are desirable for use in forecasting floods in small up-stream drainage areas. However, they are not essential. About 1917 the writer was forecasting floods in the Miami Valley of Southwestern Ohio, where the most remote parts of the watershed were not more than a hundred miles from the Central Office. Co-operative arrangements were made with the Weather Bureau observers whereby each inch of rainfall and each foot of river rise were reported by telephone or telegraph during storm periods. The observers were paid additional compensation on the basis of the number of special reports submitted. Consequently, they watched their gages and made their reports promptly. The system worked so satisfactorily that accurate forecasts of river heights could be issued well in advance of the arrival of the crest stages, not only for the principal cities of the Valley, but also for the various construction activities which were being carried on by the Miami Conservancy District.

The real value of installing recording rainfall and river gages in forecasting systems for small drainage areas lies in the satisfactory data they supply for subsequent detailed studies of hydrologic conditions. The forecasters will want to make such studies after the floods have passed, when they have time to analyze all factors carefully. During the occurrence of the storm rainfall and the arrival of the flood stages they will be too busy to make detailed quantitative analyses of run-off phenomena. At such times, the most important matter is to get the necessary rainfall and run-off reports, promptly, so that forecasts can be quickly formulated and issued in time to be of value. Consequently, telephone, telegraph, or radio transmission facilities must be available.

Mr. Hayes mentioned the desirability of conducting snow surveys in the Eastern mountains. The methods to be used in such surveys probably should be somewhat different from those used in Western United States, since the information will be used in forecasting flood-stages during the winter and spring months instead of total run-off during the spring and summer seasons. Probably the accurate measurement of the water content of the snow at selected stations throughout the eastern drainage areas will be more satisfactory for flood-forecasting work than the establishment of snow courses such as are used in the West. Measurements of the water content should also be made much more frequently than snow surveys are usually conducted.

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[57] "Failure of Castlewood Rock-Fill Dam", by Ivan E. Houk, *Western Construction News*, September, 1933, pp. 373 to 375.

John E. Field,[58] M. Am. Soc. C. E. (by letter).[58a]—In proposing five changes in the present plan of operating the U. S. Weather Bureau, the late Mr. Hayes listed snow surveys as Requirement (*c*). The method of making snow surveys proposed herein applies only to those areas where the snow banks last well into the summer and where the spring floods come from melting snows and are probably confined to the Rocky Mountains, and the Wasatch and Coastal Ranges. Even limited to those areas snow water affects the territory from the 104th Meridian, about the eastern boundary of Montana, Wyoming, Colorado, and New Mexico, to the Pacific Ocean, and especially to those areas where irrigation is practical.

In the building of the hydraulic-fill dam of the Terrace Reservoir, in the Alamosa River, in Southern Colorado, covering the years 1908 to 1912, it was necessary to know the probable run-off of the river a week or more in advance, to be able to estimate the probable peak flow from melting snow, and to know when the maximum had passed, because any quantity greater than 1 200 cu ft (the capacity of the outlet turned) must be stored or carried over the dam by flumes. By observing the snow banks, about 10 miles distant, in the canyons and gulches, or in the high mountains of the water-shed (10 000 to 12 000 ft above sea level), the writer found several that were indicative of the run-off to be expected in the days immediately following the observations. The weather conditions, of course, were taken into account in forecasting the run-off for each day as cloudy weather lessened the melting and clear days augmented it.

At the end of four years and with the records of the flow of the river, the writer felt he could predict the season's run-off quite satisfactorily, from the appearance of the snow banks in May.

Being called upon to estimate the probable supply for the City of Denver, Colo., in March, 1933, and the predictions later proving to be much in error, it occurred to the writer that a study each spring of the snow banks on the city water-shed would furnish a more reliable basis for prediction than precipitation, snow surveys, and soil moisture records, which had been used in the 1933 estimates.

The Denver Board of Water Commissioners was induced to test the theory; it furnished telescopic lenses for the cameras and designated Harry L. Potts, M. Am. Soc. C. E., to gather the data. A program was devised (since much improved by Mr. Potts) fixing the points from which photographs were to be taken, at least once each month, and then co-ordinated with stream flow and weather records.

These records are depended upon by the Board of Water Commissioners of the City of Denver, in the operation of the city system and particularly in determining whether the City can safely sell stored water to proprietors of the irrigation canals below, who are in great need of water in July and August.

It is the hope of the writer that the current research will show the method to be both simple and reliable and, supplemented by some snow

[58] Cons. Engr., Denver, Colo.

[58a] Received by the Secretary July 16, 1937.

measurements, will be valuable to all who are concerned with water from the high mountain areas. It is his wish that some Federal Department will adopt the method and that it will lead in time to the publication monthly of the probable run-off on each stream, in a manner similar to the crop reports now being furnished by the Government to the public.

NOTE.—Corrections for *Transactions* are: March, 1937, *Proceedings*, page 451, in caption of Fig. 1, change "1936" to read "1927"; and June, 1937, *Proceedings*, p. 1189, Line 17, change "173 cu ft per sec" to read "173 000 cu ft per sec."

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DISCUSSIONS

THE PASSAGE OF TURBID WATER THROUGH LAKE MEAD

Discussion

BY MESSRS. WILLIAM P. CREAGER, HAROLD K. PALMER, MORROUGH P. O'BRIEN, JOHN C. PAGE, JOHN H. BLISS, AND B. H. MONISH

WILLIAM P. CREAGER,[19] M. AM. SOC. C. E. (by letter).[19a]—The contents of this paper are of exceedingly great interest. It is hoped that more data of like character will be forthcoming for other cases.

When water containing silt in suspension enters a relatively clear reservoir, the chances of its passage through such reservoir essentially unmixed, depend upon: (1) The volume of the incoming water; (2) the slope of the bed of the reservoir; (3) the roughness of the bed of the reservoir; (4) the length of the reservoir; (5) the specific gravity of the incoming water; and (6) the size of the particles in suspension.

For the same specific gravity, the greater the volume of inflow the deeper will be the stream of unmixed water; and this factor, together with the slope and roughness, affects its velocity of flow just as in an ordinary stream. It is obvious that the greater distance the unmixed stream has to travel, the greater will be the opportunity for the suspended matter to settle.

All these conditions must be taken into consideration for a particular problem and additional information similar to that furnished by the authors will provide the basis of the constants of flow.

The specific gravity of the incoming water depends upon: (*a*) Its temperature; (*b*) its dissolved matter; and (*c*) its suspended load. The temperature of the water entering the lake may be greater than that of the lake with a resulting tendency to keep on the surface. However, the authors have shown that, for this case, the variation in specific gravity due

NOTE.—The paper by Nathan G. Grover, M. Am. Soc. C. E., and Charles L. Howard, Esq., was published in April, 1937, *Proceedings*. Discussion on this paper has appeared in *Proceedings*, as follows: June, 1937, by Messrs. O. A. Faris, Paul A. Jones, Carl E. Scofield, and Ivan E. Houk.

[19] Cons. Engr., Buffalo, N. Y.

[19a] Received by the Secretary May 27, 1937.

to temperature is negligible in comparison with the variation due to suspended load.

Although the quantity of dissolved material undoubtedly affects the specific gravity of the incoming water, there is no evidence in Fig. 2 to show that it had much influence on the discharge of silt from Lake Mead, as indicated by the following notes: (1) There was no increase in dissolved matter prior to the April discharge of silt; and (2) dissolved matter has no chance to settle, as does suspended matter. Therefore, one would expect that, if the greater specific gravity of the water due to dissolved matter were predominant, there would be some similarity between the inflow and outflow of dissolved matter. However, in August, the outflow was reduced while the inflow was increasing; whereas, in November, the reverse is true.

The writer wishes to discuss the suspended load, first, on the basis of size of particles; second, on the basis of total volume in suspension; and, finally, on the basis of the percentage of material in suspension.

Small particles settle in water more slowly than large ones. Therefore, other conditions being the same, it would be expected that turbulence caused by fine particles would find its way through the lake more readily than that caused by large particles. However, the quantity of fine particles does not seem to be the predominating feature at Lake Mead, for the following reason: The variation in size of particles entering the lake is shown in Fig. 2(*b*) of the paper. The authors state that "whenever there was an increase in load of suspended matter at the Willow Beach Station there had been a prior increase in the quantity of material less than 20 microns in diameter at Grand Canyon." However, the reverse of this statement is not the case. For instance, a considerable discharge of suspended matter from the lake occurred after the April increase in quantities of

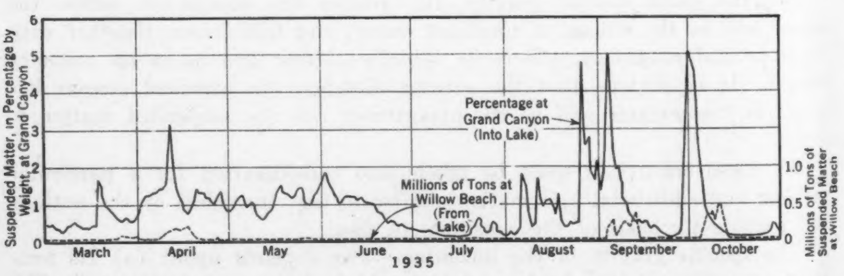

FIG. 5.—SUSPENDED MATTER, PERCENTAGE BY WEIGHT, INTO THE LAKE AND TONS OUT OF THE LAKE.

material less than 20 microns, but in June there was a still greater increase in fine particles with no resulting discharge of suspended matter from the lake. This same argument may be applied to show also that neither the total quantity of suspended matter entering the lake nor the total flow is the predominating feature affecting discharge of suspended matter from the lake.

The authors state that "there will naturally be speculation as to the reason why the high water of May and June, with its heavy silt load at Grand Canyon, did not produce a turbid discharge from Lake Mead." The writer believes that the answer is to be found in his Fig. 5, in which the percentage of suspended matter entering the lake (taken from Table 1) and the tons of suspended matter discharged from the lake (taken from Fig. 2(*c*)) are plotted. It will be noticed that there is a distinct agreement between these plottings. There was always a discharge of silt from the lake when the percentage of suspended matter entering it was greater than about 2 and no discharge of silt from the lake when the percentage was less than 2. The authors' speculation will be answered when it is noticed that the higher water of May and June with its "heavy silt load" by volume actually had a relatively small percentage of silt, and therefore, a relatively low specific gravity.

The fact that, in March, silt was carried through with only 1.7% of suspended matter whereas, in April, it did not carry through with 1.9%, may be accounted for by the fact that, in March, the percentage of dissolved matter was greater and the temperature probably less, resulting in greater specific gravity for the same percentage of suspended material.

It is not possible, with the data available, to form final conclusions; but the facts presented in the paper indicate that:

1.—It requires a certain specific gravity of the incoming water to carry the silt through the lake.

2.—At Lake Mead, the predominating feature affecting specific gravity was the percentage of total suspended load, although the percentage of fine particles and dissolved matter and the temperature may have had a minor influence.

3.—For the physical conditions at Lake Mead, it requires a specific gravity corresponding at least to that for about 2% of total suspended matter.

HAROLD K. PALMER,[20] M. AM. SOC. C. E. (by letter).[20a]—An unexpected phenomenon of scientific interest is described in this paper. Although it might be that the useful life of this large project would be increased slightly by proper operation, as the authors suggest, any possible increase in life would be small because the percentage of silt fine enough to be carried through such a long reservoir would be small. The coincidence of the increase in specific gravity of the water at Willow Beach observed from October 5 to 14, 1935, with the period of turbidity, suggests that this increased specific gravity is probably the explanation of the passage of turbid water through the lake. The authors show that the increased specific gravity is almost entirely due to the concentration of fine material, and Figs. 2 (*b*) and 2 (*c*) show that periods of turbidity follow sharp peaks of fine material observed at Grand Canyon. Therefore, a solution should be sought in the behavior of two bodies of water of different specific gravity when brought into contact with each other.

[20] Chf. Draftsman, Los Angeles County Sanitation Dists., Los Angeles, Calif.

[20a] Received by the Secretary June 24, 1937.

In collaboration with A. M. Rawn, M. Am. Soc. C. E., the writer made an extensive series of experiments on the mixture of fresh water and sea water.[21] This investigation dealt with the behavior of a lighter water introduced at the bottom of the body of heavy water which is the opposite of the conditions encountered in Lake Mead, but some of the results can be easily applied to both cases.

In the experiments with sea water it was found that the mixture of the two waters occurred in two stages, the first being a violent mixture caused by the turbulence along the plane of contact as the fresh water rose rapidly to the surface of the salt water, followed by the second stage when the somewhat diluted fresh water spread over the surface of the salt water. The term, "initial dilution", was used to express the degree of mixing that had been affected at the end of the first stage, when the water reached the surface. All the experiments revealed the fact that if this initial dilution was small (about nine parts of salt water to one part of fresh), the final mixing was very slow and the field would spread to large dimensions before disappearing at a theoretical dilution ratio of 225 to 1. It was also found that if the natural spreading of the field was interfered with in any way, such as being confined by a sea wall, further dilution was much slower and the fresh water tended to travel without much further mixing.

The authors show that about eight days after the specific gravity of the water at Grand Canyon showed an increase from 0.995 to from 1.005 to 1.008, the turbid water appeared at Willow Beach. The specific gravity of sea water is 1.026. Reducing this figure to 1.016 would be equivalent to the change in specific gravity noted at Grand Canyon and would be accomplished by a dilution of approximately two parts of salt water with one of fresh. Such a mixture would be quite stable and could be expected to continue its travel for a long distance without great dilution.

When the heavy water encounters the quiet water of Lake Mead, in a confined canyon where the cross-section increases gradually, its velocity will be reduced without turbulence, and the cross-section of the moving stream must increase in the same proportion. In the narrow box canyons which occur in Lake Mead, the bottom stream is confined and the plane of contact with light water will be narrow, thus making any mixing very slow. The velocity of the heavy water under the quiet light water is a function of the difference of specific gravity and the slope of the bottom, which latter is a constant for any given location in the lake. The velocity of the stream of heavy water will be a direct function of the difference in gravity whereas its depth will be inversely proportional to its velocity. Friction between the two classes of water complicates the problem, and the inertia of the heavy water prevents any motion until the difference in gravity attains some minimum value; but once motion is started the same inertia will maintain it until this difference has decreased to a value less than was required to initiate the movement. This

[21] "Predetermining Extent of Sewage Field in Sea Water", *Transactions* Am. Soc. C. E., Vol. 94 (1930), p. 1036.

fact would explain the movement in surges. Furthermore, since the increase in specific gravity is due to a mechanical mixture, rather than to a solution, the settling of the fine particles in the quiet water of the bottom during periods of negligible movement would tend to decrease the force required to start the flow.

The actual behavior could best be found by means of experiments with a model, and it would also be well to make regular specific gravity observations at Grand Canyon, and at Willow Beach beginning a few days after heavy water was noted at Grand Canyon. Sub-surface samples from different parts of the lake, especially at both the upper and lower ends of the wide sections, should yield interesting data regarding the difference in behavior in the box canyons and the open sections of the lake.

When the difference in specific gravity is very small, the depth of the stream of heavy material should be great and, in the valleys, it would spread well out of the old stream bed, thus increasing the plane of contact so much that mixing would be increased enough, practically, to stop the flow beyond the valley. Any model for this study should contain alternate box canyons and wide valleys to study this effect. With this information it might be possible to determine the shapes of reservoirs which would be expected to pass turbid water through them, and compare these with Lake Mead, or with any other reservoir which passes turbid water through it.

MORROUGH P. O'BRIEN,[22] ASSOC. M. AM. SOC. C. E. (by letter).[22a]—Recent theories dealing with the suspension of solid material in turbulent flow, treat the problem as one of statistical equilibrium in which the transverse components of velocity carry more material upward than downward, because of the increase of concentration in the downward direction, and thus counteract the constant settling tendency. Provided that the upward velocity components exceed the settling velocity of the material, a steady state of suspension may be attained. From this viewpoint a possible explanation of the flow of turbid water through Lake Mead may be that the greater density of the water carrying suspended material caused it to follow the bottom at a sufficiently great velocity to result in a turbulent flow capable of re-suspending the solid particles as they settled out. Thus, the inflowing stream might be held to the bottom in its course through the reservoir. This phenomenon is essentially different from the flow of cold or saline waters because the mixture is not permanent.

An approximate computation of the order of magnitude of the quantities involved may be made by assuming that the usual friction equations apply without allowance for the energy dissipated in the circulation set up in the overlying water. The resulting equation would be,

$$V = \sqrt{\frac{g_m}{k} \times \frac{p}{100} \times \frac{s-1}{s} R\,S} \quad \ldots\ldots\ldots\ldots\ldots\ldots (1)$$

[22] Prof. of Mech. Eng., Coll. of Eng., Univ. of California, Berkeley, Calif.

[22a] Received by the Secretary July 7, 1937.

in which V is the mean velocity; g_m is the gravitational force per unit mass; k is a friction coefficient; R is the hydraulic radius; S is the slope of the bottom; p is the percentage concentration; and s is the specific gravity of the material in suspension. If the layer of water is thin in proportion to its width, R may be replaced by T, the thickness. The discharge is bVT, in which b is the width. When $S = 0.0007$, $s = 2.65$, $k = 0.003$, and $b = 400$ ft, the depths and

TABLE 3.—COMPUTED VELOCITY AND THICKNESS OF SILT CURRENT.

Description	April 11	September 4	September 30
Percentage by weight	3.16	5.02	4.14
Discharge, in cubic feet per second	15 600	8 630	18 100
Computed thickness, T, in feet	21.5	12.5	22
Computed velocity, in feet per second	1.8	1.7	2.1

velocities for three days of high concentration are as given in Table 3. At best, these values are only rough approximations not only because of the simplified theory used, but also because of insufficient data. No allowance was made for differences in temperature between the reservoir and the inflowing stream, the width was assumed as 400 ft, and the slope is only an average value, and yet the results are within reason. The Reynolds number is high and these flows should be turbulent, but the vertical density gradient would probably reduce the turbulence below that which would occur in a clear stream of the same depth and mean velocity.

At a velocity of 1.5 ft per sec (which is below the computed velocities) the flow would traverse the 90-mile length of reservoir in a little less than four days. During this interval, material as small as 5 microns would settle out of still water, and the stream would lose its identity before reaching the tunnels unless the material is kept in suspension by the transverse velocities of the turbulence. The approximate average magnitude of these transverse velocities would be $0.15 \times 1.5 = 0.22$ ft per sec, or 80 ft per hr and some particles larger than 50 microns might be transported.

Using the same methods one may compute a lower limit of discharge and concentration which would maintain particles of a given size in suspension, but the result would be greatly affected by the temperature. If the incoming stream is colder than the water in the reservoir, capacity for maintaining particles in suspension is increased both because of the increased density of the water and the decreased settling velocity of the particles. If the inflow is warmer, the capacity for suspension is reduced and this effect may account for the absence of turbid outflow during the high-water period of May and June.

It is to be hoped that this paper will arouse sufficient interest among those concerned with reservoir operation that information will be obtained to form the basis of a general theory. It appears possible to develop a combination of theory and experiment which would permit computation of the flow of sediment through reservoirs with known inflow conditions, but to do so will require extensive field data.

JOHN C. PAGE,[23] M. AM. SOC. C. E. (by letter.)[23a]—Since the authors completed their paper, additional information has been obtained from field records and the observations of employees of the U. S. Bureau of Reclamation which may throw further light on this interesting phenomenon.

Storage in, and controlled release of water from, Lake Mead was started on February 1, 1935. The water discharged from the reservoir through the tunnel at river grade was clear until February 15, when the water became muddy and did not clear until the following day. This occurred again on February 19 and from February 23 to February 25. In all instances, there were storms on the lower water-shed preceding the discharge of the turbid water. The silt content in the first two runs was light, being recorded as approximately 10 000 ppm at the gaging station $1\frac{1}{4}$ miles below the outlet.

No measurements of silt content were taken for the third run, but samples were taken in the reservoir, just up stream from the inlet of the diversion tunnel, to determine the elevation of the silt-laden stratum. Sampling across the channel showed that this layer occupied the bottom of the original river channel, being about 2 ft in depth in the center. The surface of the stratum was level and was marked so definitely that a change of 6 in. in the elevation of the sampler changed the sample from clear to cocoa-colored. The dried silt from a sample taken about the middle point of this layer amounted to about one-fourth in volume of the total sample. At the time of this third occurrence, a streak of muddy water, visible on the surface, extended through the center of the reservoir for approximately 3 miles below Boulder Canyon, clear water appearing on both sides. Undoubtedly, it extended a greater distance, but was not visible. Boulder Canyon is 20 miles up stream from the dam.

Turbid water again flowed from the reservoir on March 24 and April 21. The temperatures of the water at the outlet of the tunnel increased from 52° F to 63° F during this period. The silt content varied from about 3 100 to 11 400 ppm, measured at the rating station below the dam and checked closely with the silt measurements taken at Willow Beach. Nine samples were taken at different depths for each reading. The silt content was found to be uniform for all samples, indicating that a thorough mixing occurred at the outlet gates. Previous to the passage of the water through these gates, and under similar conditions of silt load in the river, the silt content of the surface sample was about 78%; and the middle sample 85% of the bottom sample.

No measurements were made during the turbid releases in September and October, 1935, or from April 22 to May 1, 1936. Water was being released both from the bottom of the reservoir and through the gate-opening in the base of one of the intake towers 260 ft above the old stream bed during the April–May period. No turbid water came from the Arizona canyon wall outlet works which connected with the intake tower gate. The muddy water emerging from the low-level tunnel on the Nevada side

[23] Commr., U. S. Bureau of Reclamation, Washington, D. C.

[23a] Received by the Secretary July 8, 1937.

and the clear water from the outlet works on the Arizona side did not mix completely until after they had passed the gaging station. During the period of this flow, the temperature of the water was 50° F from the bottom of the reservoir, 58° F from the outlet works, and 55 to 57° F at the gaging station.

Recently, samples have been taken between the dam and the up-stream coffer-dam, and another set approximately 4 000 ft up stream from the dam. The silt stratum near the dam has its surface at about Elevation 650, whereas the silt surface farther up stream is about Elevation 695. The question has been raised concerning the possibility of silt flowing over the coffer-dam. The top of the up-stream coffer-dam is at Elevation 720, the reservoir bottom at approximately Elevation 615 near the dam, and the bed of the old river channel about Elevation 640. It is possible that the surface of the silt stratum was at one time higher than the top of the coffer-dam and since that time has lowered, due to the settling of the silt when undisturbed.

Temperature measurements recently taken near the dam at the time of the silt sampling show an abrupt rise of from 10° F to 20° F near Elevation 650. From this elevation downward the higher temperature is maintained until the bottom is reached. Measurements of specific gravity and electrical conductivity also show pronounced increases near Elevation 650. A brief summary of these data is given in Table 4.

TABLE 4.—CHARACTERISTICS OF WATERS OF LAKE MEAD MIDWAY BETWEEN UP-STREAM INTAKE TOWERS, MAY 21, 1937.

Elevation, in feet, above mean sea level	Clarity	Temperature, in degrees Fahrenheit	Conductivity (ohm reciprocal)	Specific gravity
1 066*	Clear	65.7	0.00128	1.0010
690	Clear	48.5	0.00141	1.0013
665	Clear	48.4	0.00141	0.0014
655	Clear	50.1	0.00142	1.0014
640	Muddy	60.1	0.00166	1.1732
620	Muddy	63.5	0.00161	1.1787

* Surface

Although there is not sufficient information available to arrive at definite conclusions concerning the phenomenon, it is evident that, under certain conditions, silt-laden water will pass through a reservoir without intermingling, although the length of the reservoir is as much as 100 miles. The silt that passes through Lake Mead is colloidal in character and differs greatly from the type normally carried by the stream. Under normal conditions the silt is abrasive, dark in color, and so heavy that 98% of it will settle out in a 2-hr period. The silt that has passed through the reservoir has a clay-like consistency, is lighter in color, and is difficult to remove from the water even by filtering. It is the writer's opinion that this lighter material comes from localities where the soil is predominantly clay and contains a high percentage of salts. The material

is carried to the river, generally, by cloudbursts. Its ability to pass through the reservoir seems to be derived from its colloidal and chemical characteristics.

The abrupt change in temperature at Elevation 650 in the waters near the dam may be due to the silt stratum, but the increased temperature is possibly due to the presence of warm springs. These springs, having a temperature as high as 130° F, were encountered in the canyon during construction. The heavy layer of colloidal clay in suspension may have an insulating effect which tends to delay rapid temperature diffusion.

John H. Bliss,[24] Assoc. M. Am. Soc. C. E. (by letter).[24a]—The passage of silty water through a reservoir of clear water in a more or less definite stratum, with little diffusion of the two, is a phenomenon which has not received much attention but which merits considerable study. Messrs. Grover and Howard have presented a very interesting paper on its occurrence through the waters of Lake Mead, and have suggested the possibility of utilizing such phenomena to reduce silt accumulations within the storage area of reservoirs.

Since its completion in 1915, there have been frequent passages of silt through the Elephant Butte Reservoir of the Rio Grande Project, in New Mexico, most of them occurring in the summer and fall when the flow of the Rio Grande was comparatively low and the torrential discharge of tributaries above the reservoir was relatively great.

The silt-laden waters that seem to be chiefly responsible for this phenomenon are contributed by two large intermittent tributaries, namely, the Rio Puerco, which drains an area of about 5 000 sq miles on the west side of the Rio Grande and empties into it about 45 miles above the head of the reservoir; and the Rio Salado, which drains an area of about 1 500 sq miles also west of the river and enters a few miles below the Rio Puerco. As their names imply ("Puerco" meaning "soiled" or "dirty," and "Salado" meaning "salty"), both streams carry extremely silty water of high saline content during periods of run-off. Analyses of the waters of the Rio Grande at the San Marcial Gaging Station at the head of the reservoir indicate that these tributaries may carry 150 tons or more of silt and a concentration of total dissolved salts of more than 3 tons per acre-ft of water discharged. It is interesting to note that these two streams, particularly the Puerco, have their sources immediately to the east of those arid regions drained by the Little Colorado River and the southern branches of the San Juan River, which latter streams the authors believed to be the chief contributors of the finely divided silt causing the turbid flows through Lake Mead. In this connection the authors' reference to silt passage through the Zuni Reservoir on the head-waters of the Little Colorado may also be cited.

The phenomenon of the difficulty with which two waters of different density may unite is of rather common occurrence. Any one who has seen

[24] Engr., State Engr.'s Office, Santa Fé, N. Mex.

[24a] Received by the Secretary July 27, 1937.

a muddy torrential flow enter a stream of clear water has probably observed that the two flows may often be readily distinguished for considerable distances down stream. The San Acacia diversion dam of the Middle Rio Grande Conservancy District is constructed across the Rio Grande about two miles below the mouth of the Salado and ten miles below the Rio Puerco. The gate-tender at the dam has noted several instances when the flows of these tributaries have appeared at the dam practically unmixed with the main river discharge. On one occasion, the line of demarcation between the two flows was described as quite noticeable, being almost as distinct as if they had been separated mechanically. It was further stated that such tributary waters might appear on the right, or entrance, side of the channel, or they might pass under the main stream and appear on the opposite side.

Since 1931, officials of the Rio Grande Project and the Division of Western Irrigation Agriculture, of the Bureau of Plant Industry, U. S. Department of Agriculture, in co-operation, have obtained and made complete analyses of the waters of the Rio Grande at several stations, including San Marcial above the reservoir, Elephant Butte immediately below the dam, and Leasburg diversion dam about fifty miles below. Although the saline content of the water is the primary consideration in these analyses, incidental silt determinations have been made at the first two of these stations. Periodic sampling at these points, together with an endeavor of the U. S. Bureau of Reclamation to keep a daily record of all appreciable silt passages through the lake, provide the means of studying some of the details of these occurrences. L. R. Fiock, Project Superintendent, has records of turbid flows through the reservoir for somewhat less than half the years since its construction. There is reason to believe, however, that there may have been other occasions when tributary waters have passed through the reservoir unobserved. At such times the quantity of silt carried may not have been sufficient to be noticeable, the clue to such passage being an increase in salt content of the water released from the reservoir.

During the six years that complete analyses have been made of the waters passing Elephant Butte only five periods out of more than twenty major tributary discharges were recorded as silt flows through the reservoir (see Table 5). However, as the sampling period was monthly (since 1924, weekly), the periods between the dates of sampling are long enough so that no records have been obtained during periods when silt flows of short duration might have passed through the gates. As an example, in 1931, monthly samples only were taken at Elephant Butte, except that in September a two-day flow of silt through the reservoir was recorded. During the latter part of September and October of that year there were at least four tributary discharges which might have caused turbid discharge. At Leasburg Station, where eight samplings were taken during that period (silt determinations not being made), all but one sample showed an increase in total dissolved salts carried. On October 12, the salt concentration of the water had increased 100%, the increase on other days of sampling being less marked. The data are not conclusive, because the discharge from the

TABLE 5.—DATA REGARDING TRIBUTARY DISCHARGES TO ELEPHANT BUTTE RESERVOIR, 1931 TO 1936

Dates	(a) San Marcial Record				(b) Records below Reservoir	
	Silt, in tons per acre-foot	Salt, in tons per acre-foot	Tributary Discharge		Elephant Butte dates	Remarks
			In cubic feet per second	Percentage of river discharge		
September 17–20, 1931	..	2.0	800	30	September 22–23	Average silt, 55 tons per acre-ft.; salt, 1.42 tons per acre-ft. (85 tons per acre-ft., normal).
September 25–October 23, 1931	..	..	*	..	....	No record at Elephant Butte except sample of October 5, which showed trace of silt and slight increase in salt content; increased salt content at Leasburg through most of period (no silt record).
July 13, 1932	..	..	1 800	45	....	No samples taken below dam.
July 25–29, 1932	..	..	800	50	August 1	Trace of silt and slight salt increase recorded.
August 18–22, 1932	..	..	1 500	75	....	No samples below dam during period.
August 29–30, 1932	..	..	3 500	90	....	No samples below dam; water clear at Elephant Butte, September 5.
June 18–25, 1933	48–124	0.98–1.83	3 800	65	June 23–30	Silt, 0–67 tons per acre-ft.; salt, 0.85–1.38 tons per acre-ft. (80 tons per acre-ft. normal). Increased salt content in water at Leasburg during period.
July–August, 1933	..	..	Three rises up to 2 000	..	....	No samples during period.
September 14, 1933	148	1.96	Peak of 14 000	..	....	No samples below dam.
August 28–29, 1934	..	..	6 500	100	....	Elephant Butte sampled September 1, and September 8; no record of silt passage.
September 25–26, 1934	..	1.5	2 900	100	....	Elephant Butte sampled September 29; no record of silt passage.
August 4–6, 1935	84	2.20	3 400	55	August 9–10	Average silt, 55 tons per acre-ft.; average salt, 1.64 tons per acre-ft.; (0.75 ton per acre-ft., normal).
August 21–22, 1935	170	2.14	5 700	85	August 25–26	Average silt, 21 tons per acre-ft.; average salt, 1.10 tons per acre-ft. (0.70 ton per acre-ft., normal).
August 30–31, 1935	130	1.85	4 400	100	....	Not sampled below dam during period.
September 27–30, 1935	71	1.54	2 300	60	October 5	Silt, 106 tons per acre-ft.; salt, 1.51 tons per acre-ft. (0.75 ton per acre-ft., normal).
July–September, 1936	..	..	†	..	....	No published record of any silt or salt flows through reservoir during period.

* Three periods of flow—about 75% of total discharge of river.
† Four rises of from 1 000 to 5 000 cu ft per sec from Rio Puerco; those from Rio Salado are unrecorded.

reservoir was sharply reduced about September 19 and stopped entirely on October 5, thereby changing considerably the regimen of flow at the Leasburg Station; but the indications are that enough saline tributary flow found its way through the reservoir to increase the salt concentration of the water below the dam appreciably. Whether this water was silty or even murky is not recorded, but at least, there was not enough coloration so that turbid discharge was noted and silt samples were taken at the outlet gates.

Since the discharges of the Salado and Puerco always involve increased salt as well as silt concentration of the river water, it is impossible to state to what extent each of these two factors contributes toward this phenomenon. As the authors have suggested, it is probable that, because of the greater quantity carried, silt has much the more pronounced effect of the two in increasing the specific gravity of the waters. There appear to be instances, however, in which flows have passed through Elephant Butte carrying increased quantities of salts and but very little or no silt. Although data are insufficient for proof, it seems probable in such cases that the silt gradually settles from the influent in its passage through the lake, leaving only the dissolved salts to maintain the increased density of the solution and pass through the gates.

As Mr. Fiock[25] has inferred, temperature apparently has little effect upon the specific gravity of the turbid influent. The average temperature of the waters near the bottom of Elephant Butte is about 57° F, whereas the average summer temperatures of the discharges into the reservoir are several degrees higher—in the 60's or 70's. The effect of temperature, therefore, must be slight in comparison with other factors, as its tendency is in a direction opposite to that of the observed phenomena.

W. F. Resch, Project Hydrographer, has informed the writer that soundings were made during one period of silt passage which indicated that the flow beneath the clear waters of the reservoir followed a definite stratum with little mingling of the two layers. As the outlet gates are at the bottom of the dam, it is probable that the silt flows do occur on, or close to, the flow line through the lake.

The data, although not definite, indicate that it takes the turbid water four to five days to find its way through the reservoir, a length of 30 to 35 miles, suggesting an average velocity under water of 0.5 ft per sec, or less.

The conclusions to be drawn from a study of the passage of silt through the Elephant Butte Reservoir would seem to be: (1) That any discharge into the reservoir whose waters are of sufficient density, due to very fine suspended silt (and also to salt), will potentially pass through it in an essentially unaltered condition; (2) those flows which are below this minimum will tend to pass through the lake, but will gradually lose their identity and may or may not appear at the outlet gates; and (3) under certain conditions, discharges may find their way through the reservoir, progressively leaving their silt load behind, and may appear below the dam with

[25] *Transactions* Am. Geophysical Union, 1934, Pt. 2, p. 472.

only an increased salt content to mark their passage. These conclusions are tentative and are subject to confirmation by further investigation and study. They are substantially in accord with the authors' findings, however. The bearing of Conditions (2) and (3) upon the passage of appreciable quantities of silt through reservoirs is slight.

The suggestion that, by proper attention to gate location and the approach conditions thereto, turbid waters might be encouraged to pass through reservoirs and thereby reduce accumulations of silt within their high-water lines, is one which should receive considerable study. It is related to, and, must be investigated in conjunction with, the problem of silt conservation and the aggradation of stream channels.

If a reservoir could be operated so that peak releases could be made whenever turbid waters were available at the outlet gates, considerable quantities of silt might be prevented from settling in the reservoir area. Under ordinary operating conditions, however, it is unlikely that enough change in releases could be made economically to remove much additional silt above that ordinarily carried through the gates. Furthermore, as no greater concentration of silt can be passed through a reservoir than appears in the influent above, the problem resolves itself into one of whether the inflowing water or the reservoir volume usurped by the deposited silt is the more valuable. Under certain conditions, the disposal of even part of the silt burden of a turbid stream behind retention or storage dams would seem to be advantageous, particularly in those arid Western States where consumption of large quantities of water for irrigation has resulted in the serious problem of rapidly aggrading stream channels which threaten the very existence of agriculture along their banks.

A compilation of data pertaining to the quantity of water above and below Elephant Butte Reservoir since 1930, showing conditions at the time of appreciable tributary inflows to the reservoir, is given in Table 5. The data are not sufficient to show the sources of the rises, but those listed are believed to have come chiefly from the Rio Puerco and Rio Salado. The dates of such tributary flows, their average discharges in cubic feet per second, and their percentages to total river flows as listed, are approximate values for comparative purposes only, the discharges being so variable and the time element so uncertain that exact figures are difficult to obtain. The data from which comparisons may be made of changes in silt and salt quantities in their passage through the reservoir are not good, either, because of the extreme variations of these quantities during the various stages of tributary flow. The need for additional and more definite information is evident.

B. H. Monish,[26] Esq (by letter).[26a]—The failure of fluids of different densities to mix when flowing in contact with each other at appreciable relative velocities has been recognized as a problem for many years, but it is only comparatively recently that active interest has been shown in the

[26] Assoc. Engr., National Bureau of Standards, U. S. Dept. of Commerce, Washington, D. C. Publication of discussion approved by the Director, National Bureau of Standards, U. S. Dept. of Commerce.

[26a] Received by the Secretary July 30, 1937.

phenomenon. This is particularly true in the field of applied hydraulics. The authors of this paper are to be commended on a clear presentation of carefully collected data. Workers in fluid dynamics have made mathematical analyses of the stability of streams of superposed fluids of different densities, and it was along the lines suggested by one of these analyses that the writer attempted a study of the data presented in this paper.

As a criterion of mixing between any two superposed fluids of different densities, when the density and velocity vary continuously from one fluid to the other, L. Prandtl[27] has given the expression:

$$\theta = \frac{-g\dfrac{d\rho}{dy}}{2\rho\left(\dfrac{du}{dy}\right)^2} \qquad (2)$$

in which θ = a dimensionless coefficient which exceeds a certain value when no mixing occurs; $\frac{d\rho}{dy}$ = the density gradient between the fluids; $\frac{du}{dy}$ = the velocity gradient between the fluids; ρ = the density of the fluid at that element at which $d\rho$ and du are taken; and, g = the acceleration due to gravity.

This criterion cannot be applied directly to the problem at hand, since the density and velocity gradients are not obtainable from the data. Hence, it is necessary to modify the criterion so that it will be expressed in terms of observed quantities. Linear velocity and density gradients were assumed over some characteristic (but unspecified) length, L, obtaining instead of Equation (2), the expression,

$$\theta = -\frac{gL}{2\,U^2} \times \frac{\Delta\rho}{\rho} \qquad (3)$$

or, modifying the criterion by the factor, $-\frac{1}{2}$,

$$\theta = \frac{gL}{U^2} \times \frac{\Delta\rho}{\rho} \qquad (4)$$

in which U is the mean relative velocity between the two fluids and $\Delta\rho$ is the difference in density between the current and the still water. Equation (4) will be recognized as the product of a Froude number and a density ratio, two familiar dimensionless quantities.

Since L is not determinable from the data, it was postulated that it is some dimension similar to Prandtl's "mixing path", or the distance through which a particle of fluid moves into the surrounding fluid when effecting a transfer of energy, and that this distance is dependent on the mean velocity, U, of the current, and the kinematic viscosity, ν, of the liquid composing the current, as follows: $L \sim \frac{\nu}{U}$, or,

$$L = k\frac{\nu}{U} \qquad (5)$$

[27] "Einfluss stabilisierende Kräfte auf die Turbulenz", von L. Prandtl, Vorträge aus dem Gebiete der Aerodynamik und verwandter Gebiete, p. 1, Aachen, 1929.

in which k is a numerical constant. This expression merely amounts to assuming that the Reynolds number for this phenomenon is constant. Substituting Equation (5) for L in Equation (4) and again absorbing the constant in θ:

$$\theta = \frac{g \nu}{U^3}\left(\frac{\Delta \rho}{\rho}\right) \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (6)$$

in which $\Delta \rho$ = the density difference between the stream and the still water in the reservoir; ρ = the density of the lighter liquid (reservoir water); and, ν = the kinematic viscosity of the lighter liquid.

Again taking advantage of the fact that all observations were made in the same reservoir and on the same liquids, Equation (6) can be further simplified by absorbing the constant quantities, g and ν, in the constant. Recognizing that any particle which is entirely borne, surrounded, and supported by a liquid over the range of action studied should be considered a part of that liquid and as affecting the properties of that liquid, $\frac{\Delta \rho}{\rho}$ could be computed from the data given, but it is sufficient to note that this ratio is expressed with sufficient accuracy over the range of values of n considered by the expression, $\frac{\Delta \rho}{\rho} \sim n$, or,

$$\frac{\Delta \rho}{\rho} = k_2 n \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (7)$$

in which n = the silt load, in percentage by weight, of the total sample; and k_2 = a numerical constant. This leaves only U to be converted into the form of some quantity given in the data studied.

In any given channel, U will be dependent on the flow, the exact proportionality being unknown, but unimportant for immediate purposes, since the order in which the values of the final criterion and of the critical number arrange themselves will be unchanged by any relation that may logically be assumed. As a matter of convenience, the relation was assumed that would hold if the flow were in accordance with the Froude law, $Q \sim U^3$; or,

$$Q = k_3 U^3 \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (8)$$

in which Q is the flow of the river, in cubic feet per second; and k_3 is a numerical constant. Substituting Equations (7) and (8) in Equation (6) (absorbing the constants, k_2 and k_3 in C):

$$C = \frac{n}{Q} \ldots\ldots\ldots\ldots\ldots\ldots\ldots\ldots (9)$$

in which C is a numerical criterion of mixing.

Ten peak flows were selected from Fig. 2 of the paper and C was computed by Equation (9). The results are given in Table 6. It will be noted that whenever there was a silt flow through the reservoir, C was greater than 200×10^{-6}, and, in each case, when there was a peak flow at Grand Canyon which was not followed by a silt flow through the reservoir, C was

less than 140×10^{-6}. Thus, the critical value of C for the reservoir condition in 1935 was between these values. The critical value of C for the final conditions of a full reservoir, or for some other reservoir, will not necessarily lie between these same limits, but the same form of criterion will hold.

TABLE 6.

Date (1935) (1)	Q, flow at Grand Canyon, in cubic feet per second (2)	n, percentage of silt carried at Grand Canyon (3)	$C \times 10^6$ (4)	Unmixed current through Lake Mead (5)	Date (1935) (1)	Q, flow at Grand Canyon, in cubic feet per second (2)	n, percentage of silt carried at Grand Canyon (3)	$C \times 10^6$ (4)	Unmixed current through Lake Mead (5)
March 18	7 440	1.72	230	Yes	June 16	102 000	1.09	10.7	No
April 11	15 800	3.16	200	Yes	August 6	13 000	1.82	140	No
May 18	25 100	1.52	60.6	No	August 26	13 000	4.81	370	Yes
May 24	31 600	1.48	46.9	No	September 4	13 000	5.02	386	Yes
June 2	50 200	1.90	37.8	No	October 1	17 700	5.09	288	Yes

An analysis of recorded silt flows through Elephant Butte Reservoir, New Mexico, for 1919, 1923, 1927, 1929, 1931, 1933, and 1935 shows that the same form of criterion applied to that reservoir for each of these years except 1929. The critical value of C found for this reservoir is approximately twice that found for Lake Mead. The cause of this discrepancy is being made the subject of further study, but undoubtedly this difference is due in part to the different sizes and settling rates of the silt particles entering the two reservoirs. A river that carries a large percentage of fines in its silt load would not drop as much of its load on entering a reservoir as one carrying a smaller percentage of fines and, hence, would maintain a density great enough to cause a silt flow through the reservoir with a comparatively small total silt load.

The criterion as used is dimensional and is not unique, and, therefore, is not a true criterion in the strictest sense; nevertheless, it is felt that the significance shown by the ratio, $\frac{n}{Q}$, in the cases tested is not entirely fortuitous.

As the bottom gates of Boulder Dam are now closed and, in all probability, the quantities of water discharged will never be great enough to raise the silty water to the outlet towers in easily detectable quantities, it would be necessary to detect a silt flow through the reservoir in some other manner, and there is no reason to believe that such flows will not occur in the future, the muddy water forming a submerged pool near the dam. This might be done by lowering a photo-electric turbidimeter unit into the reservoir or (as most silt flows through a reservoir occur in the summer, when the river water is warmer than the bottom reservoir water) by lowering a resistance thermometer. If it was desired to study the application of Prandtl's theory of flow as quoted, or some of the similar theories, it would be necessary to make velocity and density determinations over cross-sections of the current and the immediately surrounding reservoir. Since, to

the best of the writer's knowledge, no device has been developed for indicating on the surface the density of water at any considerable depth, it is necessary to resort to sampling for density determinations, which would allow analyses to be made to determine what caused the density difference and from what tributary or part of the system the contributing causes arose.

The whole forms an interesting problem. Any possibility of increasing the life of a reservoir by training silt flows through it must certainly wait on a fuller understanding of the phenomenon, an understanding to be obtained only by careful observation and study of such phenomena whenever and wherever they occur. Theories and laboratory studies must be substantiated by field data before full reliance can be placed on them.

The authors mention the flow of unmixed or of partly mixed river water through Lake Mead. This is entirely in accordance with theory. As may be seen from Equation (2) the vertical distance, y, over which density and velocity gradate, may have any value from nearly zero to infinity. Conditions might exist in which a stable current would be accelerated, causing mixing until the relations of $\frac{d\rho}{dy}$ and $\frac{du}{dy}$ were such that stability was again reached. After such acceleration and mixing, stable conditions might not be reached a second time, and one would have the case of a silt flow through part of the reservoir followed by mixing and eventual deposition at an intermediate point between the river and the dam.

The author's estimate of the silt carried through Lake Mead by the recorded flows is conservative. Experiments made at the National Bureau of Standards[28] show that clear water will scour a sand bed faster than water carrying fine silt, due to the cementing action on the bed material of the fine particles carried in the turbid water. The method used by the authors of subtracting the load scoured from the river bed below the dam from the total load carried by the silty water gives results which are low by an appreciable amount.

[28] "Experimental Study of the Scour of a Sandy River Bed by Clear and Muddy Water", by Chilton A. Wright, M. Am. Soc. C. E., *Research Paper No. 907, Journal of Research*, National Bureau of Standards, Vol. 17, No. 2, August, 1936.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

FLOOD PROTECTION DATA

PROGRESS REPORT OF THE COMMITTEE

Discussion

By Messrs. John C. Hoyt, and H. K. Barrows

John C. Hoyt,[7] M. Am. Soc. C. E. (by letter).[7a]—This progress report presents in an especially concise and clear manner many basic conditions relative to floods and the studies necessary in connection with them. As stated in the report, there is one point on which there is general agreement. That is "great floods have occurred as far back as historical data are available, * * *, and great floods will continue to recur in the future".

Among the various ways in which flood damage may be limited are: (1) The elimination from the flood areas of activities subject to damage; (2) the construction of levees; (3) the construction of impounding reservoirs; (4) the protection of drainage areas by a vegetable cover, including forests; and (5) the protection of drainage areas by soil conservation methods. The effectiveness of these various methods varies; therefore, each should be studied and evaluated carefully. In general, however, the only sure protection against floods is Item (1), "the elimination from the flood areas of activities subject to damage". A study of aerial photographs of floods of 1936 and 1937 indicates that there are many overflowing areas in which damage can be eliminated by comparatively slight relocations of activities along the river channel.

The continued development and occupancy of areas subject to floods invites future damage which is inevitable, and danger lines should be established below which additional activities should be discouraged or entered upon only at the owner's risk. Many areas subject to flood damage could be condemned and bought (and thereby all future trouble eliminated) cheaper than flood protection works can be constructed and maintained.

Note.—The Progress Report of the Committee on Flood-Protection Data was presented at the Annual Meeting, New York, N. Y., January 20, 1937, and published in March, 1937, *Proceedings*. This discussion is printed in *Proceedings* in order that the views expressed may be brought before all members for further discussion of the report.

[7] Cons. Hydr. Engr., U. S. Geological Survey, Washington, D. C.

[7a] Received by the Secretary June 15, 1937.

By the Act authorizing the construction of certain public works, etc., approved June 22, 1936, the Federal Government under certain limitations and restrictions assumes responsibility in connection with the protection against flood damage. Under this Act, it can be assumed that encroachment can be continued and that the Government will assume the responsibilities for protective works. Governmental agencies provide for fire control, safety of buildings, health control, etc. In this respect, however, building and other restrictions are set up and rigidly enforced. Restrictions in regard to developments in the flooded areas which will be subject to destruction should also be set up. The addition of such restrictions in connection with the Flood Control Act should be considered. At any rate, the Federal Government should establish, in the various areas along streams subject to floods, zones in which flood risks are known to exist.

In connection with flood projects, the possibility of eliminating activities from the danger zone should be considered in connection with other methods of protection. To date, very little has been said or done along this line.

The Committee's report calls attention to the matter of continuation of collection and publication of flood data. For nearly half a century the U. S. Geological Survey has been charged with the collection and publication of data in regard to stream flow, including records of stage. The records collected are made available by the Survey and form the principal basis not only for flood, but also for other, studies having to do with streams. The policy of the Survey has been, as far as facilities have been made available, to obtain all required data. Unfortunately, on account of inadequate financing, much needed information has not been collected. The Survey, however, has not only the Congressional authority for doing the work, but has an organization which includes thirty-eight field offices, one for practically every State. With adequate financing, all required information can readily be made available. The data that have been collected have been important factors in connection with the design of flood-control projects as well as other hydraulic works.

H. K. Barrows,[8] M. Am. Soc. C. E. (by letter).[8a]—The writer agrees with the Committee in its reiteration of its prior suggestions[9] relative to the continuation of the collection and publication of flood data; the making of an inventory of earlier great floods as far as practicable; and the making of a study of cloudburst floods. These are all desirable objectives. The future publication of flood data by the Federal Government in pamphlet form, with reference to river basins, is in accord with procedure by the U. S. Geological Survey and should be followed in general by Federal agencies.

The suggested campaign of education with reference to locating valuable property in flood zones is an important aspect of flood control to which more attention should be given. It is one practical method of reducing

[8] Prof., Hydr. Eng., Mass. Inst. Tech., and Cons. Engr., Boston, Mass.

[8a] Received by the Secretary June 11, 1937.

[9] *Proceedings*, Am. Soc. C. E., March, 1935, p. 339.

flood damages which is always within reach and no doubt should often be followed. The suggestion of a committee "to make a study of the various methods of controlling floods, with particular reference to their physical and economic limitations", has apparently already borne fruit and has resulted in the appointment of the Committee on Flood Control, for this specific purpose. Such a foundation for all flood-control planning is badly needed at present—particularly in the framing of sound methods for the determination of flood damages, both direct and indirect, and of reasonable flood-protection values from such studies. The present situation in this respect is chaotic, and its rationalization is a matter of the greatest importance in all flood-control project studies.

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DISCUSSIONS

PRACTICAL USE OF HORIZONTAL GEODETIC CONTROL

Discussion

BY MESSRS. PHILIP KISSAM, RALPH Z. KIRKPATRICK, E. B. ROBERTS, H. W. HEMPLE, J. C. CARPENTER, AND GEORGE D. WHITMORE

PHILIP KISSAM,[3] ASSOC. M. AM. SOC. C. E. (by letter).[3a]—A most important and basic conception has been presented by Mr. Sheldon. He has indicated the prime necessity of establishing the position of all horizontal survey points in relation to a single fundamental datum. This principle cannot be emphasized too strongly, and its general adoption at some time in the future is inevitable. All surveys will become in fact parts of one great survey and the inter-relationship between all survey points will be capable of quick determination. Although the advantages of such a condition are too many to enumerate herein, a moment's thought will convince the reader that every effort should be made to attain this aim. As Mr. Sheldon states, "a comparable example is in the use of geodetic levels. Their advantages are instantly admitted and they are used whenever possible, even where profile levels must be run for miles to base some project on the common sea-level datum." A common vertical datum co-ordinates only one dimension, a common horizontal datum co-ordinates the other two. It appears to be obvious that far greater utility of a similar nature will be found in the use of a horizontal datum.

Moreover, two indirect benefits will obtain when a survey is placed on a horizontal datum. It will be usually necessary to connect it with two or more known control points. With such a connection a check on accuracy will be obtained and the precision of results can be raised by adjustment. Furthermore, the permanent marking of surveys will be encouraged, as it will be at once apparent to the engineer in charge that points of such a survey will have a lasting value and it will be well worth

NOTE.—The paper by R. C. Sheldon, Assoc. M. Am. Soc. C. E., was published in April, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[3] Associate Prof. of Civ. Eng., Princeton Univ., Princeton, N. J.

[3a] Received by the Secretary June 12, 1937.

while to place monuments at the stations and to record the descriptions and data relating to these monuments. The cumulative effect of monumenting surveys on a standard datum will further advance the acceptance of the system.

Mr. Sheldon's description of the surveying methods in use in the Canal Zone illustrate pointedly the rapid adoption and the utility of such a system when it is known and available. He states that it has been used as a base for all surveying on canal construction since 1911, for control of part of the military maps, as a base for extending triangulation, for municipal construction, and for the boundaries of the Canal Zone and military and naval reservations.

Without question, the standard datum that should be adopted is the North American Datum of 1927, as advocated by Mr. Sheldon; but the method of expressing positions appears to be open to question. Thus far, only two methods have been developed which offer the requisite characteristics; both have sufficient advantages in their favor to merit careful consideration. Mr. Sheldon advocates the use of geographic positions. The U. S. Coast and Geodetic Survey advocates State plane co-ordinate systems based on definite projections especially adapted for the purpose. Such projections have already been designed by that Bureau for every State and in many States they are already in use.

Geographic positions must be computed in either case for the fundamental first and second-order triangulation; but with the adoption of standard plane co-ordinate systems, the plane co-ordinates of any geographic position can be computed according to the plane system existing in the area. Likewise, the plane or "grid" azimuth can be computed from the geodetic azimuth at any known position.

Although Mr. Sheldon's arguments advocating the use of geographic positions are sound, the arguments he has chosen against plane co-ordinate systems seem to the writer to be valid only when local co-ordinate systems are considered. State-wide systems have not been carefully weighed; in fact, they have been only mentioned by reference to *Special Publication No. 193*, of the U. S. Coast and Geodetic Survey (11).[2] In most areas, these State systems will produce a higher degree of accuracy than the method proposed. Does not the proposed system require reduction from geographic positions to plane co-ordinates, computation by plane co-ordinates, and, finally, reduction back to geographic positions with accompanying loss of accuracy?

It may be stated justly that in many commercial surveys a high degree of accuracy is frequently not of major importance; but lack of precision is not the basic objection to the system proposed. There is a certain fact which has not been given sufficient consideration in Mr. Sheldon's paper. It is a fact not susceptible of mathematical deduction nor scientific demonstration. It was the reason which caused the U. S. Coast and Geodetic Survey, finally, to adopt plane co-ordinates after a long and careful study of the question. It was the basic argument that influenced the Federal

[2] For references to figures in parentheses see Appendix I of the paper.

Board of Surveys and Maps to recommend the use of plane co-ordinates to its twenty-four member Federal mapping agencies. The fact is that engineers making surveys over a limited area or of low precision refuse to use geographic positions, and are entirely justified in so doing.

To the writer it appears that the cause for this stand is not the lack of a simple geodetic manual, nor the lack of training received by engineers as students, nor any other cause that can be remedied. If a remedy were available, the writer would recommend plane co-ordinates for the present and geographic co-ordinates for the future, because it would take so many years to change the thought in the engineering schools and in the profession that the general adoption of a common datum might be endangered. Mr. Sheldon describes in especially clear and forceful paragraphs, and carefully prepared tables, the simplicity of a certain system of geodetic reduction for small surveys or for surveys of the usual commercial precision. The recommended system can be criticized only for its lack of accuracy. His suggestions could not be better chosen as guides to develop the proper attitude toward geodetic reduction. If the writer could convince himself that geographic positions should be used for the type of surveys under discussion (very large precise surveys must usually be reduced by geodetic methods), he would still recommend the use of plane co-ordinates if only as a stepping stone to the later adoption of geographic positions.

Unfortunately, however, the increase in cost necessitated by the use of geographic positions precludes their adoption for commercial purposes. If the engineer can be shown that he will receive concrete advantages by using a standard datum, he may be persuaded to extend his survey to control monuments; but the extra advantages to him of expressing the stations of his survey in terms of geographic positions instead of plane co-ordinates are so limited, if existing at all, that it is impossible to justify any of the necessary increase in computing costs.

The State plane co-ordinate systems advocated are considerably better than the local systems mentioned by Mr. Sheldon. The State systems are based on carefully devised projections; in general they have a minimum dimension of about 180 miles, a maximum dimension frequently equal to the largest dimension of the State, and a maximum scale correction of one part in ten thousand. It appears that only a small percentage of engineers making ordinary surveys would be required to work in an area where such systems adjoined, and those few who might have to do so could utilize one of the systems throughout their survey. One of the great uses for a standard datum will be for the description of property. In fact it is the writer's hope that the demand for control for property surveys will be so great that it will ensure sufficient control for all surveys. It will be only the greatest exception when a property survey will cross from one State system to another. The boundaries of the systems have been especially arranged to reduce further the possibilities of this occurrence.

In conclusion, it is well to emphasize the fact that Mr. Sheldon has presented an important and timely paper, worthy of the careful study of

all civil engineers. He shows the importance of a universal horizontal datum and directs attention to the simplicity of geodetic reduction for ordinary surveys. However, in spite of offense to the mathematical sense that plane co-ordinates invariably produce and also in spite of the difficulties encountered when two such systems join, the accuracy and practical advantages they offer make them invaluable as a means of establishing the use of a universal horizontal datum and are probably the best method of utilizing such a datum in any case.

The question at issue could be stated as follows: Is the increased cost to the private surveyor, of computing geodetic positions for "run-of-the-mill" surveys, justified by the elimination of the errors and difficulties which may occur at the infrequent boundaries of adjoining plane co-ordinate systems? The writer believes that the cost is not justified.

Ralph Z. Kirkpatrick,[4] Esq. (by letter).[4a]—After having had an initiatory part in its development, the writer inherited, administratively, the horizontal geodetic control system in the Canal Zone, which is described accurately in Mr. Sheldon's paper.

In any work of sufficient importance the method is likely to warrant the expense of the requisite secondary and tertiary triangulation and traverses. An interesting application was in the 1911–12 rectangular Canal Zone Land Survey, a sectionalization of lands into 2-km squares. Unfortunately, when 40% of the intersections had been determined in the field, the project was abandoned due to the adoption of the then new military policy of nationalizing all lands and expropriating by purchase all settlers' lands, crops, and improvements.

The flexibility of the system has caused its use in the Canal Zone in virtually all city, cadastral, and topographical surveying (such as the Madden Road and Dam project), whether in the jungle or elsewhere, by the Army, Navy, and the Canal Administration. An interesting and early adaptation was in determining the errors in boundaries of certain Colombian rectangular land grants owned by the Panama Railroad Company. The lines had been run by magnetic bearings and measured inaccurately. Studies of the old documents were made and certain long-time accepted boundary points were tied into the geodetic control, and new controlled traverses were run through the accepted points. Eventually, a position was found for "Point A" (the initial point of beginning in the descriptions of the 1855 grants), for which all conditions were met. It was then learned that a mile-post (mentioned in the description), had been moved many years before. Comparison of the true azimuths through certain of these points revealed a compass variation of 7° 14′ east, virtually checking the variation (7° 15′) on the 1855 map; with that checked, the steps in correcting the boundaries in the field became mere routine.

[4] Chf. of Surveys, Panama Canal, Balboa Heights, Canal Zone.

[4a] Received by the Secretary June 14, 1937.

E. B. Roberts,[5] Assoc. M. Am. Soc. C. E. (by letter).[5a]—A problem is treated in this paper which is vital to surveying and mapping programs—a problem, therefore, the economic importance of which can scarcely be over-emphasized. The author is heartily endorsed in his plea for general use of geodetic control in the co-ordination of surveys. It is thought that this principle has the support of most informed engineers, and that the problem, therefore, is one of means to its end. The means advocated in this paper are open to strong criticism.

Because of its scope, the national geodetic datum is computed and defined in terms of geodetic co-ordinates. The reference surface being a spheroid, the azimuthal and distance relations of points on the surface cannot be determined by plane geometry. Geodetic computation of positions is necessary, as well as other special treatment. Triangulation of any magnitude does not achieve full value unless adjusted by least squares. The checking of geodetic and astronomic determinations in horizontal control surveys involves highly technical problems, and the differences are not always small, as stated by Mr. Sheldon.

These considerations are not pertinent to the problem, however, because the establishment of the national datum, and the performance of control surveys on a scope demanding geodetic treatment, are provided for by competent Federal and State organizations. The problem is how practicably to handle local surveys, mainly traverses, so they may be defined in terms of, or relative to, the national datum, a sufficient number of control points on that datum being presupposed to be available. Three possible methods are:

(1) Use of geodetic co-ordinates and pure geodetic computations;
(2) Use of geodetic co-ordinates and plane survey computations; and
(3) Use of rectangular co-ordinates on an arbitrary reference plane related to the national geodetic datum.

Method (1).—This method appeals to the geodesist because of its technical perfection. It is true that geodetic position computation in itself is simple enough to be understood by competent surveyors. It is the only form of geodetic computation that would be indicated in general for local surveys. Its use, however, for the computation of points through the mass of traverses comprising ordinary surveying would be laborious and generally unwarranted in view of the development of Method (3). For this reason Method (1) is difficult to popularize.

Method (2).—After stressing the technical unsoundness of plane co-ordinate surveys and the simplicity of geodetic position computation, the author tacitly acknowledges the laboriousness of geodetic position computations for traverse work, and proceeds to advocate Method (2). Its advantages are the use of the geodetic datum, and simplicity, with no laborious computations, and no artifice, such as a plane co-ordinate grid. Its shortcomings in the matter of accuracy are serious.

[5] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

[5a] Received by the Secretary June 17, 1937.

It involves linear adjustment of azimuth discrepancies and position closure errors in traverses between geodetic control points. Geodetic corrections are grouped with accidental errors for adjustment. This is illogical, because the geodetic corrections have no linear characteristics; that is, they have no accidental distribution.

Table 4 is an inadequate criterion of proportional errors, because it deals only with the special case of straight traverses. It is not an example of the worst condition ever likely to be encountered. Although it is true that most traverses would have better length distribution than this extreme example, it is also true that they would have poorer distribution of relative directions. In the straight traverse used as an example the linear distribution of closure errors tends to approximate the distribution of geodetic corrections. In a meandering or circuitous traverse this is not so. In the case of a traverse closing on its own starting point the algebraic sum of the geodetic corrections is zero, resulting in their complete disregard by the linear adjustment. A moment's consideration makes it clear that Table 4 has no meaning for other than straight traverses.

It is unnecessary to develop this reasoning further. Even if the proportional errors shown by Table 4 were critical, they would still be too large for acceptance for common use in the United States; in fact, they are greatly in excess of those found in modern plane co-ordinate projections.

Method (3).—In plane co-ordinates, linear traverse adjustments concern only accidental errors and, in this case, are logical. Position computational methods are technically rigorous. The source of error lies in the impossibility of correctly representing a spheroidal surface on a plane surface. The error, however, is definitely known, and can be kept small.

The Lambert and Transverse Mercator grids established for the various States by the U. S. Coast and Geodetic Survey make geodetic control available for use on plane rectangular co-ordinate projections the maximum scale errors of which are kept within satisfactory limits. As an example, the entire State of North Carolina was covered by a single projection, only at the very extremities of which did the scale error become slightly greater than 1:10 000. Obviously, throughout the greater part of the State the errors are much smaller. Even so, the scale error, everywhere, is known and readily corrected if necessary.

Admittedly an artifice, and an indirect approach to the problem, such a projection nevertheless is finding ready acceptance. It leaves to competent organizations the performance and computation of necessary geodetic surveys, and the definition of the control points in terms of the projection. The local surveyor has nothing new, either to learn or to do, but to accept a ready-made grid instead of a local, unrelated grid of his own. All the control points will eventually be on it, and his work will be that of plane survey computation in its simplest form. Nevertheless, it will be on the national datum.

From the foregoing it is argued that plane surveying is definitely not "out of date." It is, in fact, finding wider use in connection with the

projections described, and bringing the use of geodetic control more and more "into the picture."

H. W. Hemple,[6] M. Am. Soc. C. E. (by letter).[6a]—Geodetic control expressed on a geographic base has no superior when accurate methods of measurement, and exact methods of computation, are used. Mr. Sheldon is to be commended for endorsing it for local surveying purposes. However, he uses an approximate method of computing whereby plane surveying methods are applied to a geographical base, and the resulting data are expressed in terms of geographical positions. His field of activity is in the vicinity of the equator, and, therefore, he is able to do this without undue violence, because of the small rate of convergence of meridians, and because the area covered by his surveys is of limited extent.

Since he can do this, the author immediately assumes the premise that such a method should be generally adopted throughout the United States, and concludes that there is no need for the adoption of the plane co-ordinate systems such as have been devised recently by the U. S. Coast and Geodetic Survey for each of the States.

The term, "errors," is used rather loosely in the paper. To develop the spheroid representing the earth's surface on a plane it is necessary to apply certain mathematical corrections which are definitely known. These corrections can be applied to measurements made on the earth's surface, in order to relate them to the plane, and they are not errors in the sense of mistakes, blunders, or personal peculiarities of the observer, which apply to surveying observations. Mr. Sheldon considers these corrections, which are mathematically determined, in the same sense as errors of observation.

Table 3, entitled "Maximum Error Possible," purports to show the greatest possible errors which would occur for traverses computed by his method. Table 3 is misleading, however, because traverses do not usually extend in a straight line, but consist of a number of broken courses. He applies a correction for azimuth closures equally to all such courses. In this correction is included the effect of convergence of the meridians. This effect is a variable quantity, depending for any given latitude upon the length of line and its direction. In any given latitude, and for lines of equal length, it should be applied as a maximum to lines in an east-and-west direction, and as a minimum of no correction to lines extending north and south. On a line which bears eastward the convergence should be positive, and on a line which bears westward, it should be negative. Mr. Sheldon, on the other hand, distributes this effect, which is included in the azimuth closure, equally on all lines, even to those running in a north-and-south direction and to those lying in different quadrants. These effects of the convergence of meridians are not always small, and for traverses covering some distance the discrepancy due to their non-consideration, is likely to be considerable. A little study shows that to adopt such

[6] Hydrographic and Geodetic Engr., U. S. Coast and Geodetic Survey, Washington, D. C.

[6a] Received by the Secretary July 15, 1937.

a system with its attendant discrepancies generally throughout the United States will lead to endless confusion.

To state that no consideration should be given to the convergence of meridians, because these effects are far overshadowed by the type of surveying accomplished and the accuracy obtained, is to take a rather limited viewpoint. Surveys of an accuracy of 1 part in 10 000, or better, are being executed by engineers and surveyors constantly, and work of this order of accuracy is not exceptional. Instrument manufacturers report that their greatest demand is now for 20″ and 30″ instruments, whereas several years ago the minute transit was most generally sold. This means that the Engineering Profession is becoming alive to the utility and ease of execution of accurate surveys. Certainly as the land becomes more thickly populated, more accurate surveys will be a necessity, and this being the case, the surveying system accepted should be adapted to such needs.

In advocating the method of computing advanced in this paper, Mr. Sheldon states that loop closures should be avoided, and that the courses should be of equal length. Why adopt a system of computation where restrictions such as these are placed upon the field work? Any computing system recommended should be adapted to the various conditions to be encountered in ordinary field operations. The terrain is not always such that courses of equal length can be obtained. It is not always possible to have triangulation stations conveniently located at the ends of a traverse, and it is frequently necessary to make a loop and close on the same station. Certainly, if the conditions of the field work are such as these, the work should not be hampered unnecessarily by restrictions imposed by the method of computation.

The writer has computed three actual traverses: (1) By the method advocated by Mr. Sheldon; (2) by the method of the State plane co-ordinate system; and (3), by geodetic methods (see Table 5).

TABLE 5.—COMPARATIVE ACCURACY OF SURVEYING COMPUTING METHODS.

Traverse No.	State	Length of traverse, in feet	Criteria	(1) Method advocated by Mr. Sheldon	(2) State plane co-ordinate system	(3) Geodetic method
1	Nebraska	80 700	Azimuth difference Error of closure	9′–48″.3 1 to 4 000	0′–59″.0 1 to 11 700	0′–64″.4 1 to 11 700
2	New Jersey	27 600	Azimuth difference Error of closure	2′–14″.4 1 to 7 220	0′–47″.1 1 to 12 800	0′–49″.4 1 to 12 800
3	Nebraska	69 000	Azimuth difference Error of closure	7′–55″.2 1 to 11 300	0′–05″.7 1 to 30 200	0′–10″.3 1 to 30 200

These traverses were chosen at random. The field work was done in accordance with instructions for second-order surveys specifying an error of closure of 1 to 10 000, or better. These results show that the State plane-co-ordinate systems and the geodetic methods of computing are rigid, and that the same field observations give the same closures. If third-order specifications for field work had been adopted, the results from the method

advocated by Mr. Sheldon would give closures of less than 1 to 5 000 and the work would not conform to third-order accuracy. For this method, also, it is impossible to determine, for any given traverse, what proportion of error is introduced by the method of computation, and what is due to the field observations. If this method were adopted universally, and closures such as those illustrated were obtained for traverses between fixed triangulation stations, then lines run between stations established on different traverses, would be subject to large discrepancies, and the resulting confusion would be intolerable for any rigid control system.

If an extensive system of triangulation is required, the geodetic methods of computation should be used. These computations are not especially difficult or involved, and should be readily understood by any graduate of an engineering school, who has pursued the usual mathematical courses. The resulting data are expressed in terms of latitude and longitude. These terms can be readily converted to the plane co-ordinate system adopted for the State, and on that projection any traverse can be computed with greater ease to any required degree of accuracy than would pertain to computations of traverse by geodetic methods or by the method advocated by Mr. Sheldon.

These State systems of plane co-ordinates were adopted at the urgent request of practical engineers who wished to make use of the advantages accruing to their surveys through connection to the national net of geodetic control, but wished the mathematics of relating their surveys to such points to be as simple as possible, consistent with attaining the accuracy desired.

In devising the plane co-ordinate systems for the various States, two general forms of projection were used. For those States the greatest extent of which is in an east-and-west direction, the Lambert conformal projection was adopted, and for those States lying mostly in a north-and-south direction, the transverse Mercator projection was used. The earth's spheroid at sea level was developed on these projections for areas the extent of which was determined by the limiting scale factor necessary to bring the sea-level surface to that of the plane. In most cases this limiting correction, or scale factor, was kept less than 1 part in 10 000. This limitation necessitated more than one zone in most of the States. These scale factors vary according to the distance from lines of exact scale which, in the Lambert projection, consist of two standard parallels of latitude, and in the transverse Mercator projection of two lines on either side of the central meridian and parallel to it. For an area 158 miles in width, the maximum scale factor is less than 1 part in 10 000. The scale factor varies as the distance from a line, and not from a point of origin, as stated by Mr. Sheldon.

For any surveys within these zones which are connected to the national control net, and are expressed in terms of State plane co-ordinates, the distortion due to placing the surveys on the projection is, in most cases, less than 1 part in 10 000. If the scale factors are applied to the linear

measurements, which is not a difficult matter, then the limit of accuracy is dependent almost solely upon the standards of the field measurements.

It is true that at the junctions of the plane co-ordinate systems, there is a jump from one zone to the other. To obviate any difficulties due to traverse lines extending from one zone to another, a sufficient overlap is provided at the junctions, so that any traverse may be placed on either one zone or the other. It is a simple matter to convert the co-ordinates of one zone to those of the other at the point of overlap.

In making use of plane co-ordinate systems for the computation of traverse surveys, a connection is made at each end of the line to triangulation or traverse stations previously located on the national system of control surveys; or the traverse may be a loop and tie to the same initial point at the end of the survey. The positions of the stations to which connections are made are expressed in terms of x and y-co-ordinates with a grid azimuth to a definite point near-by, all on the plane co-ordinate system adopted for the State. The grid azimuth is carried through the traverse from the starting point to a closure at the station at the end of the traverse. The azimuth closure is then distributed equally among the angles of the traverse. The linear measurements are reduced to the horizontal, and if they are above about Elevation 500 they should also include the correction to reduce to sea level. If the maximum of accuracy is desired, the scale correction should also be applied. As previously stated, in most cases, this scale correction is less than 1 part in 10 000 and may be ignored if the accuracy desired does not exceed that criterion. The survey is then adjusted by the method of "latitude and departure" which is so generally known to surveyors and engineers.

The United States is not alone in the adoption of the plane co-ordinate systems for general surveying purposes. Such projections have long been in use in many European countries. Most of these countries are thickly settled and land values are high. Methods of a high degree of accuracy are required for property surveys. These countries have found a plane co-ordinate base adequate for their needs. Should not this country benefit by the experience of others? As the population of the United States becomes greater and land prices increase, surveying standards pertaining to property must be on a higher plane than has generally prevailed to this time. Plane co-ordinate systems, mathematically determined and related to the spheroid, to which field surveys can be referred by utilization of the simple methods of plane surveying, are entirely logical and consistent, and furnish a simple and readily understood method to which engineers and surveyors may turn with confidence that the accuracy desired will be obtained.

These systems are finding ready acceptance among engineers and surveyors. Two States—New Jersey and Pennsylvania—have already adopted laws legalizing the use of the State plane co-ordinate system for property description purposes. Several other States are also considering similar laws.

As to Mr. Sheldon's conclusion that plane surveying is out of date, if by this statement he means relating surveys to a State plane co-ordinate system, the writer takes issue with him. These systems are mathematically logical, and the surveying data expressed thereon are consistent among themselves. Surveyors need have no hesitancy in relating their surveys to such systems with assurance that, if the recommended methods of computing are followed, the accuracy of their data will be limited only by the standards of the field measurements.

J. C. CARPENTER,[7] M. AM. SOC. C. E. (by letter).[7a]—Any argument for the universal use of horizontal geodetic control should receive the unanimous endorsement and active support of the entire Engineering Profession. This basic scheme for reference of surveys is the only satisfactory method, and its general adoption seems so logical that it is difficult to understand the indifference displayed by the majority of civil engineers. Before engineers can attain the objective of the universal use of horizontal geodetic control they must enlist, not only engineers, but administrators, executives, and legislators, in a campaign to promote the use of this extremely valuable datum of reference. Engineers are the only members of society who can be brought to realize the value of horizontal control, and the responsibility for this educational work will fall on the shoulders of a small group who are enthusiastically determined to carry the campaign to its logical conclusion. Papers of the type presented by Mr. Sheldon are necessary to bring the subject to the attention of engineers throughout the country.

Perhaps the "wish was father to the thought" in Mr. Sheldon's statements that the main triangulation net of the U. S. Coast and Geodetic Survey is nearing completion. About 65 000 miles have been completed in the field and partly computed and published. The final spacing of 25 miles between arcs, will require 117 000 miles. Hence, the actual work is scarcely half finished. True, the projection of the remaining cross-traverses will be easier and less expensive, but there will be some work necessary on the older nets and a large volume of computation and adjustment required so that a conservative estimate places the project now (1937) at about 50% complete. The only way to complete the remainder of the work is an educational campaign among engineers, first of all, and an insistent demand from members of the profession for the completion of the net to a 25-mile spacing.

Similarly, Mr. Sheldon's optimistic statement that some States have begun to tie their extensive highway surveys to the national net is questioned. A large mileage in North Carolina has been tied to the network. Texas made a start on one survey but has never finished that one project and seems indifferent to the entire method. If the chief engineers of the forty-eight State Highway Departments of this country would spend enough time on a study of the possibilities contingent upon the use of geodetic control for highway surveys, and then give the scheme a fair trial in the

[7] Senior Highway Engr., U. S. Bureau of Public Roads, Fort Worth, Tex.

[7a] Received by the Secretary August 2, 1937.

field, they would leave a permanent legacy of basic control for all planning operations, all mapping procedure, cadastral surveys, and engineering surveys of every character which would be as valuable as the transportation facilities they provide, and, undoubtedly, its permanency would be equal to, or in excess of, the surfaces now being constructed. Geodetic co-ordinates tied in to the equator and to Greenwich meridian, converted to x and y-co-ordinates for ready use by the most imcompetent land surveyor, spaced along the highways, monumented, and the co-ordinates stenciled in the monuments, will bring the use of this system to a practicable possibility.

Mr. Sheldon has correctly stated the reasons engineers fail to use the network as a control, even if they find a monument within usable distance of their survey, but there is no excuse for the modern engineer to avoid the use of the control, for the Coast and Geodetic Survey has developed a simple and practical method of conversion from spherical to plane co-ordinates and when the x and y-values are known, it becomes a matter of the use of familiar latitude and departure surveying. O. S. Adams[8] has written a clear and convincing description of the procedure that has been developed by the Coast and Geodetic Survey for the engineers of the United States to use in making their surveys of permanent value. Mr. Sheldon has listed publications that may be consulted by interested surveyors. There are two very important publications of the Coast and Geodetic Survey, which are not mentioned, namely, *Special Publication No. 194*, "Manual of Traverse Computation on the Lambert Grid," and *Special Publication No. 195*, "Manual of Traverse Computation on the Transverse Mercator Grid." These excellent publications outline in clear and concise form the procedure to be followed in using plane co-ordinates. Every detail is covered and each possible problem is illustrated by examples. Plane co-ordinate tables for the State may be obtained from the Coast and Geodetic Survey and any engineer who may decide to use the Geodetic network can obtain complete information and detailed instructions from that agency.

Mr. Sheldon has described the system in use on the Canal Zone and his description admirably illustrates the difficulties, probabilities of error, and complications in closure that may be expected when an isolated area is to be covered by a system of control. This is the best possible argument for the adoption of the plane co-ordinate control adopted by the Coast and Geodetic Survey for all surveys in the United States. This Canal Zone system of control is not suitable for general use in the United States. Without doubt, the national geodetic net will ultimately be used as a control for all surveys, but the realization of this condition will be brought about through the use of the plane co-ordinate systems developed by the U. S. Coast and Geodetic Survey for all the States of the Union. Conformal conic projections flattened to a plane will provide simple, easily used surfaces and allow engineers to use ordinary surveying methods and the co-ordinates will be right-angle x and y-distances, in feet. When the procedure is so simple and easy to apply there is no excuse for any engineer

[8] "State-Wide Systems of Plane Co-Ordinates," *Civil Engineering*, January, 1937, p. 33.

to become frightened and fail to take advantage of the control survey. With the completion of the triangulation system and publication of the positions, together with the plane co-ordinates based on these new State systems, the adaptation of this system for all surveys will be very much facilitated.

Air-mapping has developed rapidly during the twenty years, 1917–37. No doubt many important improvements in methods and technique are just "in the offing." There are numerous agencies operating throughout the United States and the pictures being obtained are undoubtedly superior to any existing maps; but the horizontal control that is necessary to make these pictures display, accurately, the correct and true position of the topography is not in existence and there is some question as to the value of these pictures in the permanent mapping method for the entire country. It is a case of "the cart before the horse" and, in this instance, the cart may be found to be of limited value.

The geodetic network must be completed by the agencies of the National Government. Its completion is the first important step in the development of a logical plan for mapping, planning, construction, and maintenance of public and private works. Engineers are competent to explain the immense value and vital necessity of this work and, therefore, the "Practical Use of Horizontal Geodetic Control" warrants the attention of all members of the profession, and its application should be actively encouraged for all surveys.

George D. Whitmore,[9] M. Am. Soc. C. E. (by letter).[9a]—The proposition set forth by Mr. Sheldon, to compute all horizontal surveys on the spherical co-ordinate base, unquestionably has considerable merit. He convincingly urges engineers to use the ideal spherical co-ordinate basis for preserving survey results, and for rendering such results universally usable. It is believed, however, that the procedure he proposes is too far ahead of the times. Perhaps the end result which he describes could be better accomplished by getting to it more gradually—by easy stages or steps.

Many surveyors and engineers of the present time, for example, although understanding generally the mathematics and theoretical value of rectangular co-ordinate systems as computing and plotting bases for survey results, nevertheless do not clearly realize all the every-day practical advantages to be gained by using even this simple type of co-ordinate system. Hence, a majority of engineering surveys to-day are executed and used without benefit of any kind of a co-ordinate base—spherical or plane. From the standpoint of the engineers' education, therefore, it might be preferable to stress, for a few years at least, the importance and advantages of any system of co-ordinates, rather than to try to plunge at once into full use of geographic co-ordinates. After all, the use of co-ordinates, whether they are spherical or rectangular, is only a means to an end. The primary purpose of any co-ordinate system is to show by a simple mathematical expres-

[9] Chf., Surveys Section, Eng. Service Div., TVA, Chattanooga, Tenn.

[9a] Received by the Secretary July 30, 1937.

sion the relationship of each point to all other points of the system. The units of measure used, whether feet, meters, or degrees of arc, are only a means of writing this expression.

Most engineers appreciate the necessity for some kind of co-ordinate basis in constructing maps covering large areas, especially if several map sheets are involved. If the maps cover extremely large areas, such as a State, the necessity of spherical co-ordinates is not questioned. If the maps cover smaller areas, such as a city or county, the chances were, until quite recently, that the maps would have been based on some kind of a local plane co-ordinate system. Thus, in strictly mapping operations, the need for a co-ordinate basis is not questioned. Either through inertia or lack of complete understanding, however, engineers have not as a body carried this same reasoning into their engineering surveys. It seems apparent to the writer that what is good practice for mapping is also good practice for surveying.

Perhaps the principal reason why engineers have been slow in using co-ordinate systems in surveys is their realization of the shortcomings of local plane co-ordinate systems, so well explained by Mr. Sheldon. It so happens that there is now available, however, a type of plane co-ordinate system which overcomes many of the disadvantages of such local systems, but which retains many of their advantages. This is the State Plane Co-Ordinate System developed and published for each State by the U. S. Coast and Geodetic Survey in 1934. So much has already been written regarding these State-wide plane co-ordinate systems that there is no need to repeat the descriptions herein. It might be stated in summary, however, that these systems extend over large areas without serious distortion in either distances or positions. Such distortion as does exist in distances or co-ordinates is exactly known from the relation between the spheroid and plane, and, hence, can be determined instantly in case more precise distances or positions are reqired. These systems, on the other hand, have all the advantages of local plane co-ordinate systems in simple, easy computations. They have also the principal advantage of spherical co-ordinates in that they are related directly to all the basic control surveys of the country; it is a matter of only a few moments of computing to convert plane co-ordinates to geographic, and *vice versa.*

Thus, it seems apparent that the State-wide plane co-ordinate systems, representing a nice compromise between the universal geographic and the local plane systems, will be the basis, if any, used by engineers in the computation of their surveys during the next several years. Regardless of the real merit of getting all surveys on the universal spherical base, it will take several years and much more propaganda than a few papers in the technical journals to convert engineers to the actual practice.

The experience of the surveying organization of the Tennessee Valley Authority, which adopted the State-wide plane co-ordinate systems soon after their publication, is gratifying. Until that time, all routine reservoir surveys were computed on local plane co-ordinate bases. All valley-wide

projects (usually extending over parts of seven States), such as basic planimetric maps, topographic maps, and regional maps, were plotted on the geographic co-ordinate base. Extensive mapping projects similar to these latter are still plotted on the geographic co-ordinate system, with the principal control surveys being computed in terms of spherical co-ordinates. The traverses serving as detailed control for such maps are computed either on the State plane system or the spherical system, whichever happens to be most convenient. Since such map sheets show the projection lines for both systems, either basis may be used in plotting.

All reservoir maps and surveys, however, since they extend over smaller areas (each reservoir being usually entirely within one State), are now computed on the State plane co-ordinate bases. Such surveys include traverse control; large-scale topographic maps of dam sites, camp sites, and town sites; cadastral surveys and maps for land acquisition; transit-tape traverse of the final reservoir property boundary; ranges for measuring future silt deposits; transmission line surveys, and many other types. In the computing unit, where a large volume of computations are handled, using both geographic and plane systems, there is practically unanimous opinion that the State-wide plane systems fulfill a long-felt need, that they must displace local plane co-ordinate systems, and inevitably will be used by practical engineers rather than the more cumbersome and difficult spherical system.

AMERICAN SOCIETY OF CIVIL ENGINEERS

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DISCUSSIONS

PRESSURES BENEATH A SPREAD FOUNDATION

Discussion

BY MESSRS. O. K. FRÖHLICH, DONALD W. TAYLOR, AND JACOB FELD

DR. ING. O. K. FRÖHLICH[25] (by letter).[25a]—The practical significance of Part II of Professor Krynine's valuable paper consists in the emphasis, placed upon the influence of the rigidity of structures on the distribution of soil pressures in the contact surface. Equation (16) represents the result of the treatment of the "classical" case of the stress distribution at the base of a circular and absolutely rigid disk, with a symmetrical load, placed on the horizontal surface of a semi-infinite, elastically isotropic mass. Boussinesq[10] has given this result in a very intuitive manner, which, in the writer's opinion, is worth mentioning here for two reasons: First, because it represents, for the practical engineer, a mnemo-technical means of reproducing Equation (16); and second, because it may be used to compute the stress distribution in the two-dimensional case. According to Boussinesq the unit stress at the base of the rigid circular disk (of radius, r) at any point, $c\,r$, of the contact surface is obtained by distributing the total load, $P = p\,r^2\,\pi$, equally over a hemisphere ($2\,r^2\,\pi$), which is assumed to be erected over the base circle of the disk, and by taking that part of this load, which is cut from the hemisphere by a vertical cylinder, having the unit surface at the given point as the base. It is easily seen that, in the center, where $c = 0$, the stress is $0.5\,p$; at the edge where $c = r$, it becomes infinitely great and at any point, $c\,r$, Equation (16) is satisfied.

In the case of a rigid strip (width, $2\,b$) of infinite length (the case of plane stress), a hemi-cylinder, having a diameter, $2\,b$, must be introduced instead of the hemisphere. The total load, $\bar{P}$, per unit of length equals $2\,b\,p$ and the load per unit of the cylinder surface is then $\frac{2}{\pi}\,p$. In a manner similar to

NOTE.—The paper by D. P. Krynine, M. Am. Soc. C. E., was published in April, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[25] Cons. Engr., The Hague, Holland.

[25a] Received by the Secretary June 1, 1937.

[10] "Application des Potentials à l'Etude de l'Équilibre et du Movement des Solides Élastiques", Paris, 1885, p. 158.

the three-dimensional case, the formula for the pressure, p_z, at a distance, cb, of the axis of the loaded strip, becomes,

$$p_z = p \frac{2}{\pi \sqrt{1 - c^2}} \qquad (31)$$

which is analogous to Equation (16).

H. Borowicka[26] and A. Habel[27] have dealt with the influence of the rigidity of foundation slabs and strips on the distribution of the soil pressures in the contact surface. One of the characteristic results of these investigations is that, under equally distributed loads of slabs and strips for any degree of rigidity, the pressures under the edges are infinitely great, which is a consequence of the assumption, made in these theoretical computations, that Hooke's law remains valid beyond any limit. In this respect, the practical question arises as to the value that the pressure may attain under the edge of a foundation.

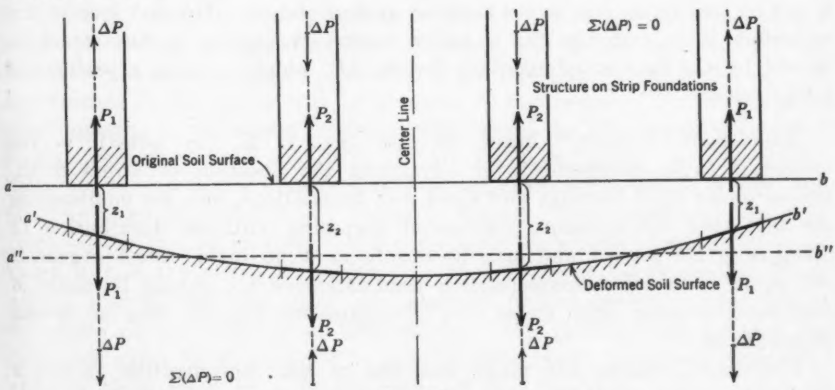


FIG. 16.

An attempt to answer this question of the magnitude of the critical stress under the edge of a foundation was presented by the writer in 1934.[28] All the aforementioned theoretical computations of the distribution of the soil pressures under loaded plates or strips, having a certain degree of rigidity, relate to very simple cases which, in practice, are very seldom encountered.

To show how the practical problem of determining the influence of the rigidity of a given structure on the distribution of the soil pressures may be attacked, the following example will be considered: In Fig. 16 the four foundations, symmetrical about a center line, consist of infinitely long strips, supporting a rather rigid structure. The loads, P_1, P_2, may be determined as usual under the assumptions applying to unyielding foundations. This is a problem of pure statics, the solution of which is well known. The assumption that the structure is infinitely long simplifies the computation inasmuch

[26] "Druckverteilung unter einem gleichmässig belastetem elastischen Plattenstreifen, welcher auf der Oberfläche des elastisch-isotropen Halbraumes liegt," von H. Borowicka, International Assoc. for Bridge and Structural Eng., Second Congress, Final Rept., 1937.

[27] "Die auf dem elastisch-isotropen Halbraum aufruhende, zentral symmetrisch belastete elastische Kreisplatte," von A. Habel, *Der Bauingenieur*, 1937, H. 15–16.

[28] "Druckverteilung im Baugrunde," Equation (5), p. 84.

as it reduces the three-dimensional, to a plane-stress, problem. The equations, given in the paper and in the monograph mentioned[28] enables the designer to determine the settlements, z_1 and z_2, if the concentration factor and the compressibility of the soil are known.

As the originally straight surface line, *ab*, of the soil changes into the curve, *a′ b′*, it is evident that the distribution of the total load, ΣP, which originally was P_1-P_2-P_2-P_1, is changed by the interaction of structure and underground. How the supplementary system of forces, $\Sigma(\Delta P) = 0$, acts on the structure and on the ground, is shown in Fig. 16. The unknown value of ΔP must satisfy the condition that the deformation of the structure becomes identical with the deformation of the surface line of the soil. Especially, if the structure is absolutely rigid, the supplementary system of forces, $\Sigma(\Delta P) = 0$, must flatten the curve, *a′ b′*, into a horizontal straight line, *a″ b″* (dotted in Fig. 16). As the settlements, due to the action of the supplementary forces, vary directly with ΔP, the numerical work involved is not so tedious as one would suppose at first glance. The last step of this procedure is to compute the so-called secondary stresses in the structure, caused by the four supplementary forces, ΔP, which is again a problem of pure statics.

Donald W. Taylor,[29] Assoc. M. Am. Soc. C. E. (by letter).[29a]—The author is to be commended for advancing the question of stress distribution under rigid footings and rigid mat foundations, and for emphasizing the fact that the common practice of assuming uniform distribution of pressure is not logical and may be unsafe in some cases. Most engineers will agree with his recommendation that extensive use should be made of pressure-measuring cells under new structures to furnish data on actual distributions.

Professor Krynine has made free use of the concentration factor, a parameter which should be adopted only when the extent to which it is applicable is held clearly in mind. The same point may be said to apply to Dr. Fröhlich's work[3] which, however, contains many valuable conceptions for a student of this subject. Fröhlich has shown that under the edges of a rigid footing on the surface of a sand deposit a condition of plastic flow must exist for which the concentration factor will have a maximum value of perhaps 6 or 8. Under the footing, progressing from the edge to the center, this factor will decrease from point to point and also it will decrease with increasing depth. Thus, the concentration factor at different points for the given case may vary as much as from 8 to 4 and with this in mind it may be seen that diagrams such as Fig. 9 may be very misleading.

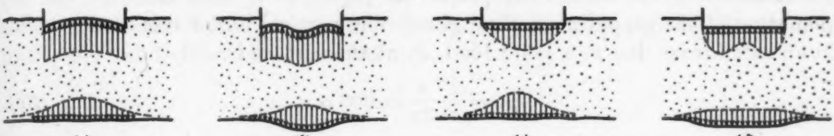
The author makes the statement that consolidation of deeper strata has no effect on the interaction between a structure and the earth mass on which it rests; that the entire system may be assumed to settle uniformly.

[29] Research Associate, Soil Mechanics, Dept. of Civ. and San. Eng., Mass. Inst. Tech., Cambridge, Mass.

[29a] Received by the Secretary June 9, 1937.

[3] "Druckverteilung im Baugrunde," 1934.

This can no more be possible than that the middle support of a rigid continuous beam can settle without increasing the load which the outer supports must carry. In Fig. 17(*a*) consider the loaded area to be the surface of a sand deposit where there is a complete lack of rigidity, and for simplicity let it be loaded uniformly and let the load be applied instantaneously. Before the underlying layer has had time to compress



(*a*) (*b*) (*c*) (*d*)

FIG. 17.

appreciably the settlement, which will occur entirely within the sand, will be as shown; and the stress thrown to the buried layer, according to the usual conceptions of spreading out of stress, will be as indicated. From this distribution of pressure on the buried layer, it is evident that the compression in this layer will be greatest at the center. The loaded area must undergo a settlement approximately equal to this compression and after a period of time will have settled about as shown in Fig. 17(*b*). Since the surface stress distribution does not change, the stress at the compressible layer is probably not radically altered. For comparison, in Fig. 17(*c*), let the footing be rigid. When suddenly loaded the stress distribution under it and the stress transmitted to the clay will be as shown. It is evident that in this case there is a tendency toward greater settlements at the center of the loaded area, and yet the rigid footing requires that the settlement must be uniform. The only possible result is that as settlement proceeds there must be a readjustment of the two stress distributions of Fig. 17(*c*), since the stress distribution at the compressible layer is caused by the stress at the loaded area and also must be such as to give uniform settlement. Fig. 17(*d*) is a rough indication of probable distribution for the rigid footing after a period of time.

JACOB FELD,[30] M. AM. SOC. C. E. (by letter).[30a]—Because of the clarity of exposition as well as the completeness of the paper, the author's contribution to the study of soil mechanics is one of outstanding merit. The limitations of the results are carefully stated and no universal solution of a very important problem is claimed.

The broad application for Equations (1) and (6) is possibly open to criticism. The derivation of Equation (1) is based on a summation of stresses resulting from strains. As long as the summation of strains resulting from stresses does not exceed that at the elastic limit of the isotropic mass, methods of superposition are permissible. Conclusions 1 to 5 are worth careful study and attention.

[30] Cons. Engr., New York, N. Y.

[30a] Received by the Secretary July 27, 1937.

It might be proper to point out that the stresses, s, are acting in the direction of the radius vector, joining the point of loading with the point at which the stress is being determined; also, that the graphical method described gives the stress at any one chosen point in the earth mass and must be repeated for each point at which knowledge of the stress is desired.

As a practical problem, the vertical intensity of loading is required. A continuation of the basic formulas of the paper, as is done later for the explanation of the graphical method gives: The vertical unit stress at any point in an earth mass due to a point load, P, at the surface is,

$$p_z = \frac{P}{z^2}\frac{n}{2\pi}\cos^{n+2}\alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (32)$$

The vertical unit stress at any point in an earth mass due to a uniform linear load, $\overline{p}$, at the surface is,

$$p_z = \frac{\overline{p}}{z}\frac{n_1}{\pi}\cos^{n+1}\alpha \ldots\ldots\ldots\ldots\ldots\ldots\ldots (33)$$

Equations (32) and (33) are derived from Equations (1) and (6) of the paper. For various values of n and α, tables based on the values for n_1 listed in Table 1, can be prepared for ready use, as shown in Table 2. By the use

TABLE 2.—VALUES OF $\frac{n}{2\pi}\cos^{n+2}\alpha$ AND $\frac{n_1}{\pi}\cos^{n+1}\alpha$.

	(a) VALUES FOR EQUATION (32)				(b) VALUES FOR EQUATION (33)			
n	$\alpha = 0$	$\alpha = 15°$	$\alpha = 30°$	$\alpha = 45°$	$\alpha = 0$	$\alpha = 15°$	$\alpha = 30°$	$\alpha = 45°$
3	0.477	0.401	0.232	0.085	0.637	0.545	0.359	0.160
4	0.637	0.517	0.269	0.080	0.751	0.632	0.366	0.133
5	0.796	0.625	0.292	0.070	0.850	0.690	0.358	0.106
6	0.955	0.723	0.303	0.060	0.939	0.735	0.343	0.083
7	1.114	0.815	0.306	0.049	1.019	0.771	0.323	0.064
8	1.27	0.897	0.301	0.039	1.090	0.797	0.299	0.048

of Table 2, upon an assumed value of n, and with a given loading, stress conditions at various depths along radial lines 15° apart can be computed and stress intensity contours plotted.

Except in unusual cases, or at shallow depths, there is seldom any necessity to go into great details such as are outlined in the example illustrated by Fig. 6. Taking the data of that example, a very rapid approximate computation, using a slide-rule only, can be made as follows: Divide the area into three areas by the horizontal and vertical lines through Point X, and locate the center of gravity of each area:

$A_1 = 79 \times 60$; the center of gravity is 39.5 right and 30.0 up; α is

$$\tan^{-1}\sqrt{\frac{39.5^2 + 30.0^2}{100^2}} = 26° \; 20'; \text{ and, } A_1 \cos^5\alpha = 2\,740.$$

$A_2 = 55.25 \times 60$; the center of gravity is 27.63 left and 29.0 down; α is

$$\tan^{-1}\sqrt{\frac{27.63^2 + 29.0^2}{100^2}} = 21° \; 50'; \text{ and, } A_2 \cos^5\alpha = 2\,280.$$

$A_3 = 65.75 \times 60$; the center of gravity is 32.96 left and 31.0 down; α is

$$\tan^{-1}\sqrt{\frac{32.96^2 + 31.0^2}{100^2}} = 24° \; 20'; \text{ and, } A_3 \cos^5\alpha = 2\,480.$$

(The foregoing values are obtained from the correct location of the centers of gravity of the two trapezoids. It is close enough even to assume the centers 30 ft up and 30 ft down from the axis.)

The summation of $A \cos^5 \alpha$ is 7 500, and the vertical unit pressure 100 ft below Point X is 0.89 ton per sq ft. This may seem to be in considerable error, the author's value being 0.78, but at a depth of 100 ft, the weight of the soil is approximately 5 tons per sq ft, and the totals are, therefore, practically identical. Much closer approximations, of course, can be determined by further subdivision of the loaded area.

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DISCUSSIONS

SOIL REACTIONS IN RELATION TO FOUNDATIONS ON PILES

Discussion

By Messrs. Hibbert M. Hill, and George P. Stowitts

Hibbert M. Hill,[3] Assoc. M. Am. Soc. C. E. (by letter).[3a]—A very interesting series of examples is presented in this paper and the author's deductions from his study of them are exceptionally clear cut. After reading the paper, however, the writer has an impression that Mr. Miller makes it appear that more weight can be given to pile-driving and loading tests than is actually the case. Such tests are of value for specific purposes (as, for example, to determine whether a pile will develop frictional resistance sufficient to transfer its load to the surrounding soil, or, in the case of piles reaching a hard stratum through soft material, to test the strength of the piles as columns), but in the usual case it is impossible to test load an area comparable in size to the foundation. Therefore, in the usual case, the results of test loadings, in plastic materials especially, cannot be relied upon to indicate directly the settlements to be expected in the finished structure.[4]

The author places great emphasis, and rightly, on a thorough exploration of the underground by borings. It is not a flattering commentary on the state of the art that this point requires emphasis. The writer has had experience involving the design and construction of eight locks and dams on the Upper Mississippi River. These structures were founded on piles in undergrounds containing sand, silt, and clay, complicated in some cases by rock at depths of about 15 ft. Borings by various methods, driven test piles, and loaded piles were used to explore these foundations. The period of construction of the project as a whole was of sufficient length to provide settlement data for comparison with the test results from some of the structures before the final structure was completed. At the termination

Note.—The paper by R. M. Miller, M. Am. Soc. C. E., was published in June, 1937, *Proceedings*. This discussion is printed in *Proceedings*, in order that the views expressed may be brought before all members for further discussion of the paper.

[3] Supt., St. Anthony Falls Water Power Co., Minneapolis, Minn.

[3a] Received by the Secretary June 23, 1937.

[4] For a discussion of this subject see, "The Science of Foundations," by Charles Terzaghi, M. Am. Soc. C. E., *Transactions*, Am. Soc. C. E., Vol. 93 (1929), pp. 288–291.

of this experience, the writer summarized in his notebook what he had learned about foundations. The red letter entry in this notebook is:

"By far the most important foundation information is a thorough knowledge of the material in the underground to a depth of about twice the width of the foundation; to greater depths if soft beds might exist at those depths. For all explorations use a method which will obtain a whole sample of the material, from ground surface to bottom of boring, and where compressible materials are encountered, obtain undisturbed samples for laboratory shear and consolidation tests."

An expert, it is true, can forecast future behavior quite accurately by inspection of undisturbed samples, but the expert will reserve his opinion until he has the laboratory results before him. Only the inexpert are courageous enough to proceed, where compressible materials are involved, on the basis of inspection of ordinary boring samples.

GEORGE P. STOWITTS,[5] M. AM. SOC. C. E. (by letter).[5a]—This discussion of the behavior of pile foundations in mixed soils, with its wealth of example and data, seems to the writer to make for clarification of certain basic considerations in the design of pile foundations.

In mixed soils, piles must still be primarily point bearing or primarily friction bearing. Where there is within reach an underlying stratum of sufficient bearing capacity, the nature of the superimposed strata becomes relatively unimportant, and the case may be considered as point bearing; it becomes entirely proper to resort to a spacing as close as driving conditions permit. It is then important that the piles be of as nearly a uniform cross-section as possible. Where there is no such underlying stratum within reach, which will carry the load of a normal footing within its own area, the piles must be considered as friction piles. A friction pile is one which depends on the cone of pressure to reduce the unit load at and below its point to a value capable of being supported by the available strata. If these cones of pressure intersect and overlap too much there is a waste corresponding to the overlap. Thus, in the case of friction piles, the spacing is the controlling factor.

Although the spacing called for by the Whangpoo formula (see heading "Spacing for Friction Piles" in the paper) is extreme, none the less it is based on sound logic. Spacings of 2.5 ft or 3.0 ft, perfectly correct with point-bearing piles, must usually be very wasteful when used with friction piles; 3.5 ft to 5.0 ft would represent the most usual practical answer. This will usually call for a larger foundation area; but not always, as the unit value of the piles will be higher and more uniform.

It seems to the writer that Mr. Miller has drawn the wrong conclusion from the dish-shaped settlement (see "Spacing for Friction Piles"). These necessarily reflect an overloading of the underlying substratum; and, consequently, are more affected by the size of the foundation than by the exact number of piles. The interior piles may well be given a fairly generous

[5] Chf. Engr., Inspection Div., PWA, Washington, D. C.

[5a] Received by the Secretary July 31, 1937.

spacing; crowding in more piles will have no effect one way or the other. Close spacing in the outer ring will often have the further effect of establishing a cleavage plane, so that it becomes difficult to get the full value of the supporting soil outside the pile cluster. Where one of these settlements has occurred it can be ascribed more often to the foundation being too small rather than to its having too few piles.

Where there are many piles under a footing, much can be done to make their value high and uniform by beginning to drive in the center and working out toward the edge. This throws the heaving and compacting outward where one will be relatively harmless, the other helpful. Starting at the edge makes the piles more and more difficult to drive, needlessly increases the trouble with heaving, and decreases the capacity of the piles.

All the foregoing comment involves essentially, a friction-pile problem; such phenomena can scarcely happen where the piles are entirely end bearing. It may be worth noting in this connection that tapered piles, regardless of their material, are at their best as friction piles. In an end-bearing pile, the useful column material (for load carrying as distinguished from driving) cannot be much more than double the lower half. In heaving ground it is important that the pile, as driven, have abundant tensile strength; that is to say, that cast-in-place concrete (one of the best possible materials in compressible soils) is out of place in an incompressible soil.

Much can be done to control heaving by the sequence of the pile-driving. Often there is one side of a foundation where heaving will be relatively harmless. By starting the driving on the side with the greatest resistance (the one against the bank, if there is one), and progressing back and forth over the long rows, the heaving can be mostly thrown away from the driving to where it will make the minimum of trouble. This takes more cribbing, and more moving of the driver, but will often save several times this extra cost.

Loss of bearing capacity after driving often occurs, but so does gain. Along the New Jersey shore, opposite about 50th and 65th Streets in New York City, there is a peat deposit as deep in places as 210 ft; 85-ft wooden pier piles driven with a 3 900-lb drop hammer must be watched closely lest they go below the cut-off. They will often be going down 3.5 in. at the last blow with a 10-ft drop; but the next day these same piles can scarcely be started with the same hammer. These piles are loaded successfully to 15 tons, despite the alarmingly small value given by the *Engineering News* formula. It has seemed to the writer that the nearer one comes to a pure friction pile, the less reliable are the results given by this formula.